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CAMERON STATION ALEXANDRIA, VIRGINIA

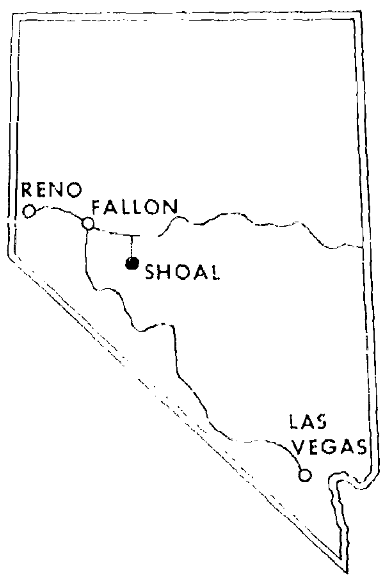
VUF-2600
FINAL REPORT

VELA UNIFORM

PROJECT SHOAL

SPONSORED BY THE ADVANCED RESEARCH PROJECTS AGENCY OF THE DEPARTMENT OF DEFENSE AND THE U.S. ATOMIC ENERGY COMMISSION

FALLON, NEVADA
OCTOBER 26, 1963



Project 1.6

IN-SITU STRESS IN GRANITE

Lucius Pitkin, Inc.

February 1964 I. H. Wright

VUF 2600

OPERATION SHOAL

PROJECT OFFICERS REPORT - PROJECT 1.6

IN-SITU STRESS IN GRANITE

F. H. Wright, Project Officer

February 1964

Lucius Pitkin, Inc.
New York, New York

ABSTRACT

The object of Project 1.6 was to determine principal stress field present in the granite at the Shoal site by measurement of secondary principal stresses in an underground alcove prior to the shot. Secondary principal stresses were calculated from in situ stress relief measurements using the borehole gage overcoring method and biaxial modulus of elasticity determinations made in the Lucius Pitkin, Inc. laboratories. Vertical stresses in boreholes in the granite adjacent to the alcove site were much less than the calculated overburden stress due to gravity. Stresses determined in situ were tectonic. The rock was fractured and faulted to extreme degree.

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INTRODUCTION

This investigation, referred to as Project 1.6, was performed by Lucius Pitkin, Inc. (LPI) for the Defense Atomic Support Agency, Department of Defense, and funded by the Defense Atomic Support Agency under Contract No. DA-49-146-XZ-244 dated 1 July, 1963.

In situ stress measurements by the borehole deformation method were performed in the tunnel, Project Shoal at Fallon, Nevada, starting on 16 September, 1963, and ending on 30 September, 1963.

Because of various difficulties encountered during the construction of the Shoal tunnel, measurements were postponed from early Summer 1963 to Fall 1963 and reduced in scope from a possible maximum of four sites to one site.

OBJECTIVES

The purpose of this investigation was to determine the magnitude and direction of the principal and secondary principal stresses in the walls, face and floor (or roof) of test adits, drifts, or alcoves in from one to four sites in the East and/or West underground tunnels extending from the shaft in granite at Fallon, Nevada. In addition, density and crushing strength of representative rock samples from each site were to be determined.

BACKGROUND

Formerly, stresses in mine tunnels and underground openings were calculated on the assumption that the stress field was due solely to the weight of the overburden, and that the stress concentration at openings could be calculated from elastic theory using data determined in the laboratory on small rock specimens tested under uniaxial loads.

By this method, only a rough approximation to the true stress condition could be obtained. It is known that tectonic stresses are frequently present which would alter the stress field. Further, elastic constants determined on small specimens are different when loads are other than uniaxial.

In recent years much research has been directed at measuring rock stresses in situ. The method employed by LPI for measuring in situ rock stresses is essentially that which has been developed by the U. S. Bureau of Mines (References 1 and 2).

This method requires the placing of a borehole gage into a $1\frac{1}{2}$ inch pilot hole drilled into the rock, taking an initial reading, then overcoring to effect stress relief with a 6 inch diameter bit while the gage is still in place. A second reading is then taken. The difference between readings, or deformation, is then related through calculation and physical property data of the particular rock to the direction and magnitude of extant principal stresses.

The borehole gage used by LPI is essentially the same as the gage described in Reference 3, except for the development of an improved insertion and retraction system incorporated on LPI equipment.

TEST SITE DESCRIPTION

LOCATION

A plan drawing of the Shoal tunnel complex and location of the test alcove is given in Figure 1; plan and elevation views of the test site alcove are given in Figure 2.

The test alcove located at 0 + 75 feet in the west drift was cut into the north wall of the drift. The site coordinates were estimated at N-1,620,760 E-556,487 from plan drawing Reference 4. The elevation of the tunnel floor had been surveyed as 3989.53 feet. Depth from the surface to the test site floor had been calculated as 1296.29 feet.

GEOLOGY

Geological description was abstracted from Reference 5. The "Ground Zero" granite is a porphyritic biotite granite with many large orthclase feldspar phenocrysts. The orthoclase crystals are up to two inches long in a medium-to-coarse-grained matrix of quartz, orthoclase and plagioclase feldspar, and varying amounts of small biotite mica flakes. Occasionally, there is some hornblende.

The granite is cut by numerous aplite-pegmatite dikes, ranging from less than one inch to more than 20 feet in width. Some of the very small dikes were encountered at the stress-relief drilling site. There are also dikes of andesite, rhyolite, and an intrusive breccia.

By its stratigraphical relationship to surrounding rocks, and from isotopic age determinations made on the biotite, the granite is considered to be of Cretaceous age.

STRUCTURE

The test site, which is located in the Sand Springs

Range, has been uplifted along a series of high angle faults which trend mainly northeast and northwest.

The northeast trending faults strike approximately N30°E and are accompanied by closely spaced parallel fracture cleavage. Some of the faults are grouped together to form fault zones. Most of the faults contain gouge and brecciated rock. Adjacent rock is commonly iron stained, bleached and propylitized. Predominate movement was vertical, though there has been some horizontal movement.

At nearly right angles to the fault system there is a second system trending more or less N50°W. These faults are accompanied by parallel joints. The majority of the aplite-pegmatite, andesite and rhyolite dikes are intruded along these faults and joints. Again, some of these faults are grouped together to form fault zones, containing gouge and brecciated wall rock. Predominate movement was vertical, in many cases estimated at hundreds or thousands of feet. There was some horizontal movement in the range estimated from tens to hundreds of feet.

THEORY AND EXPERIMENTAL PROCEDURE

IN SITU STRESS EQUATIONS

It has been shown that for a biaxial stress field, deformation of a borehole as measured during overcoring relief is related to the magnitude and direction of the applied stresses in a plane perpendicular to the hole axis.

From measurement of hole deformation at 60 degree angles of separation on stress-relief overcoring, the secondary principal stresses can be calculated as follows:

$$S + T = \frac{E}{3d} (U_1 + U_2 + U_3) \qquad (1)$$

$$S - T = \frac{\sqrt{2}E}{6d} \left[(U_1 - U_2)^2 + (U_2 - U_3)^2 + (U_3 - U_1)^2 \right]^{\frac{1}{2}} \qquad (2)$$

$$\tan 2\theta_1 = \frac{\sqrt{3}\,(U_3 - U_2)}{2U_1 - U_2 - U_3} \qquad (3)$$

where

S, T = secondary principal stresses in the plane perpendicular to the axis of the borehole, psi;

E = modulus of elasticity of the rock, psi;

d = diameter of a borehole, inches;

U_1, U_2, U_3 = borehole deformation at a 60 degree separation (60 degree deformation rosette), inches. U is positive for increasing diameter; and

θ_1 = angle <u>from</u> S <u>to</u> U_1, measured in the counter-clockwise direction, degrees.

If $U_2 > U_3$, θ_1 is between 90 and 180 degrees.

If $U_2 < U_3$, θ_1 is between 0 and 90 degrees.

If $U_2 = U_3$, and if (a) $U_1 > U_2$, $\theta_1 = 0$ degrees;

or if (b) $U_1 < U_2$, $\theta_1 = 90$ degrees.

Figure 3 is a schematic diagram showing how a borehole may deform on stress-relief overcoring, and the deformations that are measured at points 60 degrees apart.

OPERATIONAL PROCEDURE

All stress-relief drilling was performed in the conventional manner, using water as a coolant and for flushing cuttings from the holes. Overcored 6-inch core removed from the holes was marked to indicate hole designation, orientation, footage, and type of break.

Because the number of sites for stress-relief measurements was reduced from four to one, it was considered important to drill duplicate holes in all directions, time and conditions allowing. It was hoped that duplicate holes would give confirmatory data that otherwise would have been obtained from other drill sites.

Six holes were drilled in the field for a cumulative total footage of 65.75 feet. For simplification, field holes have been renumbered so that the results of the four good holes appear consecutively.

Hole 1, originally S-1-1, was drilled horizontally to a total depth of 163.56 inches S76°E into the East rib of the alcove. Usable data was obtained from a depth of 10 inches to 150 inches, but the ground from 150 inches to the end of the hole was fractured and gave no reliable results.

Hole 2, originally S-1-3, was drilled 180.31 inches into the same wall as Holes 1 and 5, 21 inches above and parallel to Hole 1. Usable data was obtained from a depth of 30 inches to 176 inches.

Hole 3, originally S-1-5, was drilled S14°W into the South rib of the main tunnel opposite the alcove by special permission after S-1-4 encountered faulted rock toward the north. Hole 3 ran horizontally 170.50 inches, penetrating a small fault zone between 55 inches and 68 inches depth. Photograph 3b shows the parallel breaks and altered edges of the rock in the fault. This hole yielded usable data in core before and after the fault zone. Because the drill blocked the haulage track, only one hole was drilled into the South rib.

Hole 4 (Up), originally S-1-6, was drilled vertically up into the roof of the alcove at the approximate conjunction of the center lines from Holes 1 and 3. It was drilled to a depth of 130.13 inches. Usable data was obtained from 20 inches to 110 inches. The core beyond this depth broke while drilling was in progress.

Hole 5, originally S-1-2, was drilled horizontally 61.25 inches S76°E into the East rib of the alcove, 21 inches (center to center) below Hole 1. This hole was abandoned because the granite was broken and shattered, and only two reliable readings were obtained in this depth.

Hole 6, originally S-1-4, was drilled 83.25 inches N14°E into the face of the alcove. From a depth of 41 inches to the end, the hole was in a zone of faulted rock, completely broken and reduced to gouge. No usable data was obtained. The hole was abandoned at 83.25 inches.

Table 1 gives the depth, elevation and bearing of each of the six holes drilled.

Photographs 1 through 6, taken by DASA personnel, show all cores recovered from overcoring operation. The cores were marked to indicate nature of the numerous fractures encountered at the various hole depths. The meaning of symbols used in marking these cores is given in Table 2.

Two to three representative samples of 6-inch hollow core from each of the good holes were taken for laboratory determination of modulus of elasticity.

STRESS-RELIEF MEASUREMENT

The procedure employed in the field for measuring borehole deformation was as follows:

1. A $1\frac{1}{2}$-inch diameter (EX gage) pilot hole, to receive a borehole gage, was diamond drilled to the desired depth interval.
2. A 6-inch diameter thin-walled pilot bit was utilized to start overcoring. Then, a 24-inch long by 6-inch diameter diamond bit was used to drill out broken or shattered granite at the mined face until relatively sound rock was reached.

3. A calibrated borehole deformation gage was inserted to a depth of 5 inches from the near end of the pilot hole. The gage was oriented to measure vertical deformation in a horizontal hole, or along a known compass direction in a vertical hole.
4. An initial "zero" reading was recorded.
5. With the borehole gage in place, the pilot hole was overcored to a depth such that further drilling produced no additional change in the gage reading. This usually occurred at a depth of about 1½ to 3 inches past the point being measured. Gage readings were recorded at each ½-inch interval of overdrilling.
6. A final gage reading was taken. The borehole deformation is the difference between the initial and the final reading multiplied by the gage factor.
7. Next, the gage was moved into the pilot hole a minimum of 3½ inches past the previous overcored depth, oriented 60 degrees clockwise from the direction of the initial reading, and the overcoring procedure repeated.

8. The gage was again moved $3\frac{1}{2}$ inches past the last overcored depth, oriented 60 degrees counterclockwise from the direction of the initial reading, and the overcoring procedure repeated.
9. This procedure of 60 degree measurements was interrupted after each two successive sets of measurement to break and remove sound core ('core pull'). It was also involuntarily interrupted each time the core broke or a natural fracture or fault was intercepted.
10. Borehole gage readings were taken until a hole depth was reached such that borehole deformation on overcoring became relatively constant, or until successful measurements became impossible owing to incompetent rock.

Photograph 7 shows the drilling rig and insertion of the borehole gage preparatory to stress-relief overcoring in a horizontal hole.

The "readings" of the borehole gage are strain units read on a standard portable electrical strain gage indicator. Basically, the strain indicator is a device for measuring the imbalance caused by a change in electrical resistance in bonded strain gages.

The change in resistance is produced by deflection of the borehole gage sensor as the pilot hole changes diameter on overcoring.

Photograph 8 shows the portable strain indicator, two borehole gages (one of which is taken apart to illustrate the cantilever beam sensor within the bullet-shaped protective housing), and accessory equipment for gage insertion and calibration.

MODULUS OF ELASTICITY DETERMINATION

Modulus of elasticity, used in the equations for calculation of secondary principal stresses, was determined by the biaxial method.

Representative specimens of hollow core 5-5/8-inch outer diameter by at least 8-inch sound length were encased in a sleeve of live gum rubber and placed within a heavy 6-inch long steel cylinder with the ends of the core protruding. O-rings were then stretched over the core ends and compressed into tapered ends of the steel cylinder by bolting on heavy steel end-rings of smaller inner diameter.

With the outer surface of the hollow core sealed at each end of the steel cylinder, the annular space outside the rubber could be filled with water and pressurized.

A borehole gage was inserted and aligned with respect to an original known orientation of the core, and an initial reading recorded. The enclosed section of core was then hydrostatically compressed in uniform increments, and gage readings were taken as pressure was increased to 3500 psi. The same procedure was performed on gradual release of pressure. Photograph 9 shows the hydraulic equipment, pressure cylinder with hollow core and borehole gage in place, and strain indicator used in obtaining the modulus data.

Modulus data were obtained on two core pieces from each hole - one from the first sound core section obtained near the face, and another from a core section at the greatest possible depth in the hole. In some holes, modulus was also obtained on a third intermediate depth core section.

Modulus of elasticity was calculated according to the following formula:

$$E = \frac{4ab^2P_o}{(b^2 - a^2)U}$$

Where U = Borehole deformation, microinches;

P_o = Hydraulic pressure applied to outside of rock cylinder, psi;

a = Inside radius of rock cylinder, inches;

b = Outside radius of rock cylinder, inches; and

E = Secant modulus of elasticity, psi.

DENSITY OF GRANITE

Density of representative rock samples was determined on NX (2-1/8-inch diameter) core samples taken from the ends of three different core holes at the site.

Density was determined by direct measurement of weight and volume.

CRUSHING STRENGTH OF GRANITE

Solid NX (2-1/8-inch diameter) core was drilled from the ends of the three successful holes. Two specimens were cut from each of these cores and the ends ground parallel in preparation for crush testing by unconfined uniaxial compression. Height of specimen was equal to diameter.

RESULTS

PRINCIPAL STRESSES AND ORIENTATION

Field readings were converted to borehole deformations by use of the appropriate borehole gage factor. This data of borehole deformations versus hole depth for the four successful holes is shown graphically in Figures 4 through 7.

The borehole deformation data was processed on an IBM 1620 computer. For this purpose a modulus of elasticity constant of 10 million psi was used. The computer results were then modified by substitution of the modulus of elasticity as determined by the biaxial method in the laboratory, to arrive at the secondary principal stresses and their orientation.

Calculated secondary principal stresses and their orientation are given graphically in Figures 8 through 11.

MODULUS OF ELASTICITY

Typical curves of hydrostatic pressure versus borehole deformation from which modulus of elasticity for the granite core could be calculated are shown in Figure 12.

These curves are reasonably straight-line functions, and the areas between loading and unloading curves are small. Table 3 gives the moduli obtained on core sections at indicated depths and concentrations for successful Holes 1 through 4.

Individual values of average modulus determined on a number of core sections tested in the laboratory varied from the overall average of all core sections tested, to an extent greater than might have been expected for a normal statistical variation. This indicated that the granite characteristics were non-uniform. Rather than use an average modulus for all stress calculations, the actual modulus determined at the point at which it was measured was used. Where the modulus data for hole depths was not determined, interpolated modulus values were used.

DENSITY OF GRANITE

The average density of three core sections was found to be 163.4 pcf. Handbook value (Reference 6) for granite is reported as 165 pcf average.

CRUSHING STRENGTH OF GRANITE

The crushing strength, or ultimate strength in compression, of the six samples of core tested is reported in Table 5. There was wide scatter in results of the samples from the same hole, but there appeared to be a significant difference between crushing strength of core from the horizontal holes as compared to the vertical up hole.

It is normal for unconfined uniaxial crush tests of rock to show wide scatter in results. Handbook values for granite building stone (Reference 6) show a range of strength from 15,000 to 26,000 psi.

DISCUSSION

The Shoal granite at the test site was so fractured into blocks of random sizes that determination of in situ stresses met with many difficulties. During diamond drill overcoring, natural fractures were encountered frequently, with maximum distance between transverse fracture planes seldom exceeding 2 feet.

Many of the blocks indicated penetration of groundwater weathering effects up to several inches from the fracture faces.

Even at depths of 12 to 14 feet from the rock faces, stress relief measurements in the overcored boreholes tended to fluctuate considerably instead of leveling out smoothly as in sound unbroken hard rock.

Holes 1 and 2 were drilled almost parallel with a fault only 6 feet in back of the alcove end face, while Hole 3 was drilled toward a large fault passing almost parallel to the drift, about 42 feet to the south.

In horizontal Hole 1 in the depth range 130 to 150 inches from the rock face, measured principal secondary stresses were of the order of 930 psi for the maximum stress S, bearing N14°E in a plane tilted down about 39° off horizontal, and 330 psi for the minimum stress T. In horizontal Hole 2, a duplicate of Hole 1, the maximum measured stress S was approximately 730 psi and T was 275 psi, in the depth range 150 to 170 inches.

In horizontal Hole 3, drilled into the south wall of the drift 90° clockwise from Hole 1, at a depth of 160 inches the maximum measured stress S was 262 psi, almost horizontal at a bearing of S76°E, while T at

178 psi was nearly vertical.

The maximum measured in situ stress S in Hole 4 was 630 psi, horizontal at a bearing of N28°E. The minimum horizontal stress of 280 psi in Hole 4 was measured 120 inches above the alcove roof, bearing S62°E. This conforms reasonably well in magnitude and direction with the maximum stress measured in Hole 3.

Using the data above as the measured secondary principal stresses, along with their known angles, the procedure for determining the stress ellipsoid 7/ used by APL, BuMines was applied.

In an x, y, z set of coordinates corresponding to S14°W, S104°W and vertical directions respectively, the normal and shear stresses were computed as:

σ_z	- (normal, vertical)	-588 and -180 psi
σ_x	- (normal, along x-axis)	-610 and -672 psi
σ_y	- (normal, along y-axis)	-260 and -300 psi
τ_{zx}	- (shear in zx plane)	-297.08 psi
τ_{zy}	- (shear in zy plane)	-11.58 psi
τ_{xy}	- (shear in xy plane)	-82.16 psi

Although the pairs of redundant values for normal stresses along the x-axis and y-axis agree within 7 and 5 percent respectively with their averages, the values for normal vertical stress vary ± 53 percent from an average of the two. Accordingly, computation stopped at this stage.

This difference between vertical stress values implies major variations 7/ in the stress field in the area where measurements were made, as might well be expected in this region of multiple faults. For example, deep in Hole 3 the location of in situ stress measurements was closer to a major east-west fault than it was to the corresponding location deep in Hole 1. There was an unavoidable distance of some 34 feet between these two points of measurement.

Inaccuracies in measurement do not appear to have caused the large difference in the redundant values for σ_z, for in situ stresses measured in the duplicate Holes 1 and 2 agreed within ± 12 percent.

Based upon the measured density of 163.4 pcf and depth of horizontal boreholes at 1291 feet below the surface, gravity loading should be 1464 psi. It appears that the test site happened to span the space between three major faults which bridged the site so that only the rubble beneath their walls produced gravity stresses. This could account for the very low and unequal values for vertical stress σ_z.

CONCLUSIONS

The maximum in situ rock stresses measured in the test area were in a plane tilted more toward the horizontal than toward vertical, dipping N-NE.

Values for vertical components of in situ stress, computed from data obtained in three orthogonal directions, differ significantly from each other and are much less than theoretical gravity stress of the overburden, indicating probable bridging effects of nearby faults.

Redundant values of vertical components which differ more than $\pm$ 15 percent from their average are outside the usable range, and completion of a stress ellipsoid determination for the heterogeneous stress field would be worthless.

In situ stresses in the test area were tectonic in the sense that they related to the fault structure rather than to homogeneous gravity loading of the medium.

COMMENT

Conclusions of this report are based upon work which was restricted to only one test site owing to mining difficulties which reduced the time allotted to Project 1.6.

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7. L. Obert; "Procedure for Determining the Stress Ellipsoid"; Correspondence; 1963.

TABLE 1. OVERCORED BOREHOLE DEPTH, BEARING, AND ELEVATION

Hole No.		Depth, inches [b]	Bearing [a]	Elevation, feet
1	(horiz.)	163.56	S75°37'E	3995
2	(horiz.)	180.31	S75°37'E	3997
3	(horiz.)	170.50	S14°14'W	3994
4	(vert.-up)	130.13	S14°14'W	3999 - 4010
5	(horiz.)	61.25	S75°37'E	3993
6	(horiz.)	83.25	N14°14'E	3997

[a] For horizontal holes, bearing is of hole axis.
For up hole, bearing is of U_1 direction (see Figure 3).

[b] Total depth of all holes drilled was 65.75 feet.

TABLE 2. CORE MARKING SYMBOLS

Symbol	Explanation
B.	Core Broke while drilling was in progress.
C.P.	Core Pull. Core was broken off by the operator for removal.
F. or N.F.	Naturally occurring fracture.
xxxxxxxx	Crack.
———→	Indicates top of core and points into hole.
(e.g) 37.5"	Numbers refer to depth of hole in inches.
(e.g.) S-1-1	Shoal - Site 1 - Hole 1. Core designation: Project, Site No., Hole No.

TABLE 3. MODULUS OF ELASTICITY OF STRESS-RELIEF CORES

Hole No.	Depth in Hole, inches	Modulus, 10^6 psi: Orientation [a] 0^o	Modulus, 10^6 psi: Orientation [a] 90^o	Modulus, 10^6 psi: Average
1	59	7.21	7.41	7.31
1	143	7.58	7.93	7.76
2	5	5.83	7.38[b]	6.30
2	5	5.46	6.50	
2	63	6.48	6.85	6.64
2	63	6.48	6.74	
2	136	7.60	7.96	7.84
2	137	7.60	8.19	
3	40	7.62	7.91	7.77
3	114	6.56	6.34	6.45
3	150	5.98	5.94	5.96
3	151	6.11	5.81	
4 (Up)	27	7.05	7.66	7.36
4 (Up)	84	6.66	7.23	6.95
4 (Up)	117	6.58	7.33	6.96

[a] Orientation of borehole gage in horizontal core with respect to vertical.
In the vertical (Up) hole, 0 degrees has a bearing of S45°46' E.

[b] Values of E modulus in brackets are averaged together.

Average = 7.03×10^6 psi
Variance = σ^2 = 0.403
Standard Deviation = σ = 0.635

TABLE 4. CRUSHING STRENGTH OF GRANITE

Hole No.	Depth in Hole, inches	Crushing Strength, psi
2 (horiz.)	218	16,600
2 (horiz.)	223	10,700
3 (horiz.)	173	12,700
3 (horiz.)	191	10,500
4 (vertical-up)	133	8,900
4 (vertical-up)	151	6,900

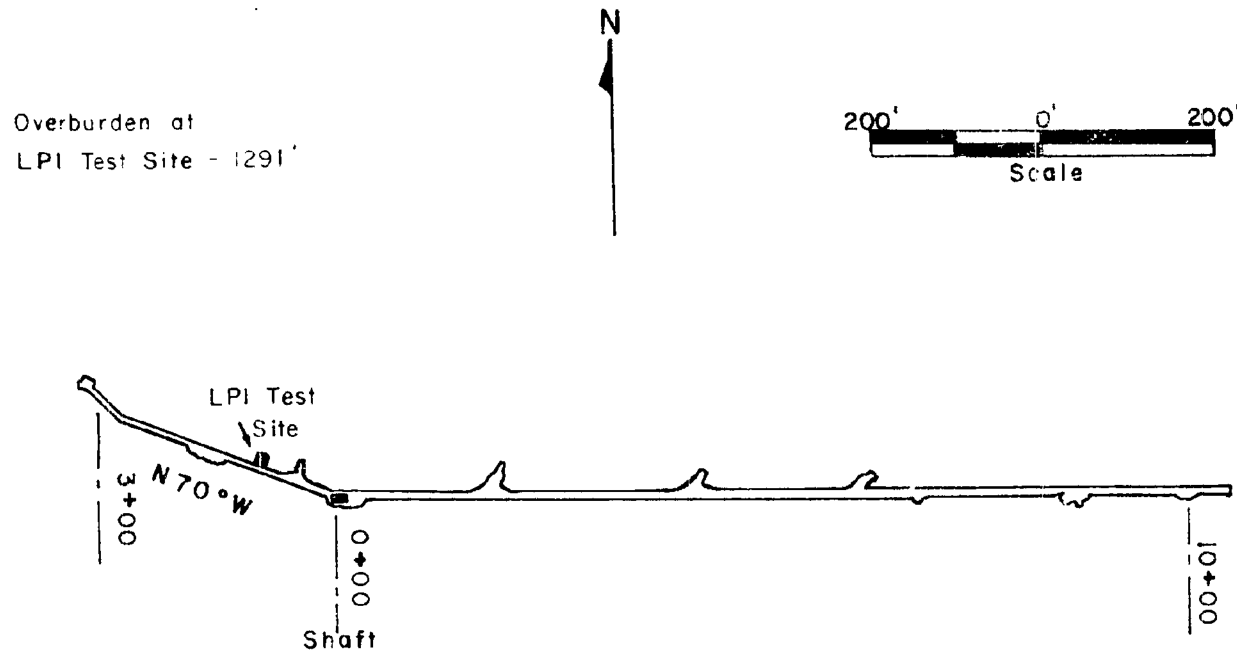

Fig. 1-Site Location for Stress Relief in West Drift

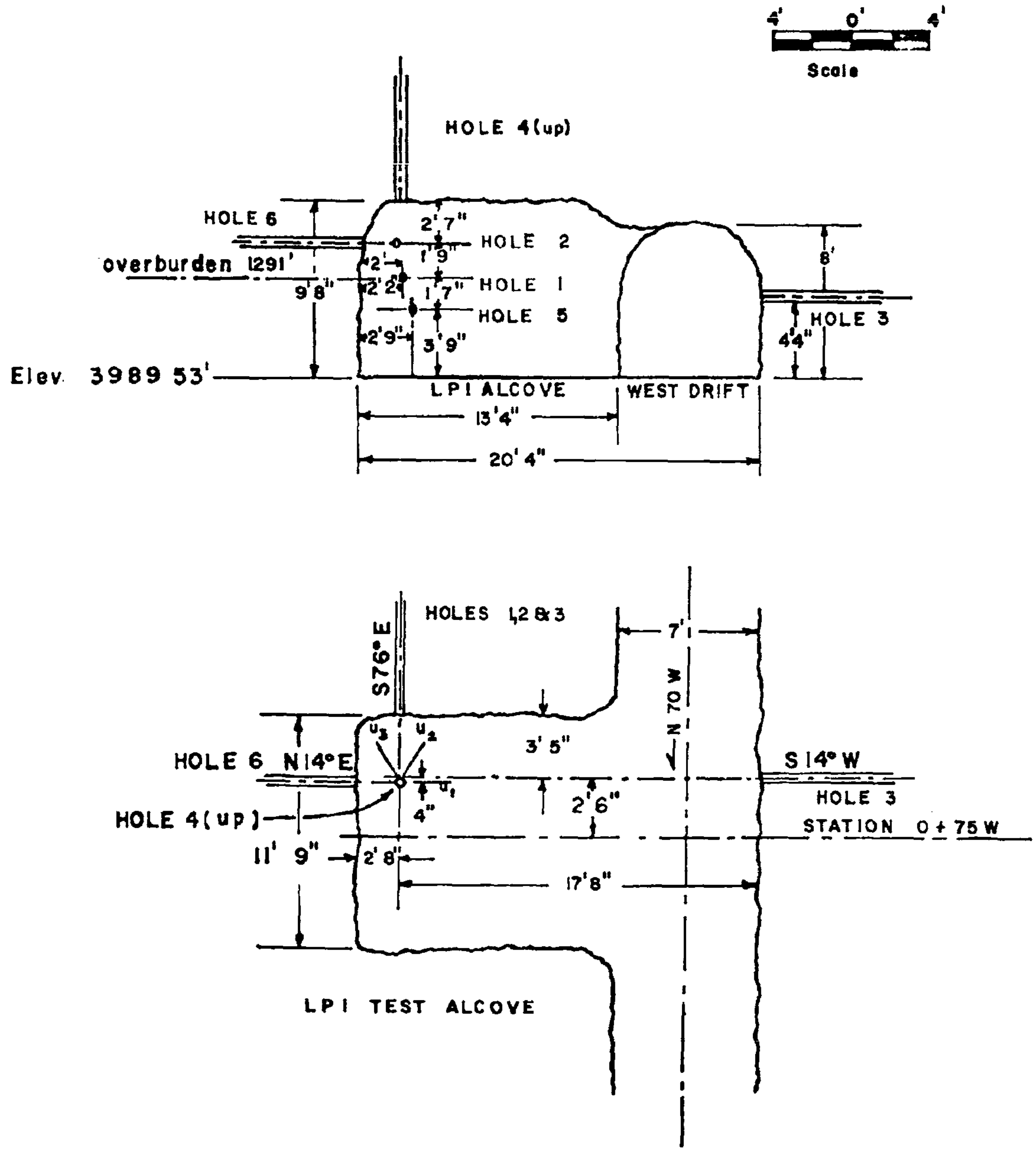

Fig. 2 - Detail Plan and Elevation of Drilling Site

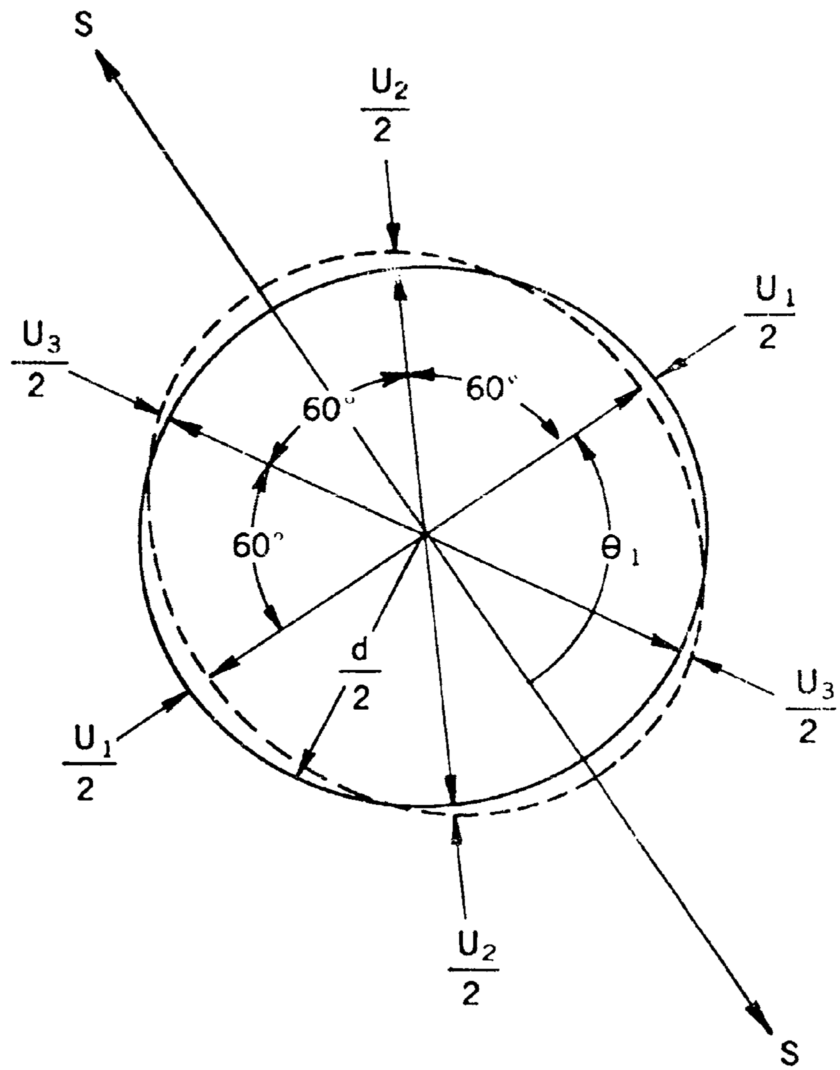

Figure 3
Deformation of a Borehole
60° Rosette

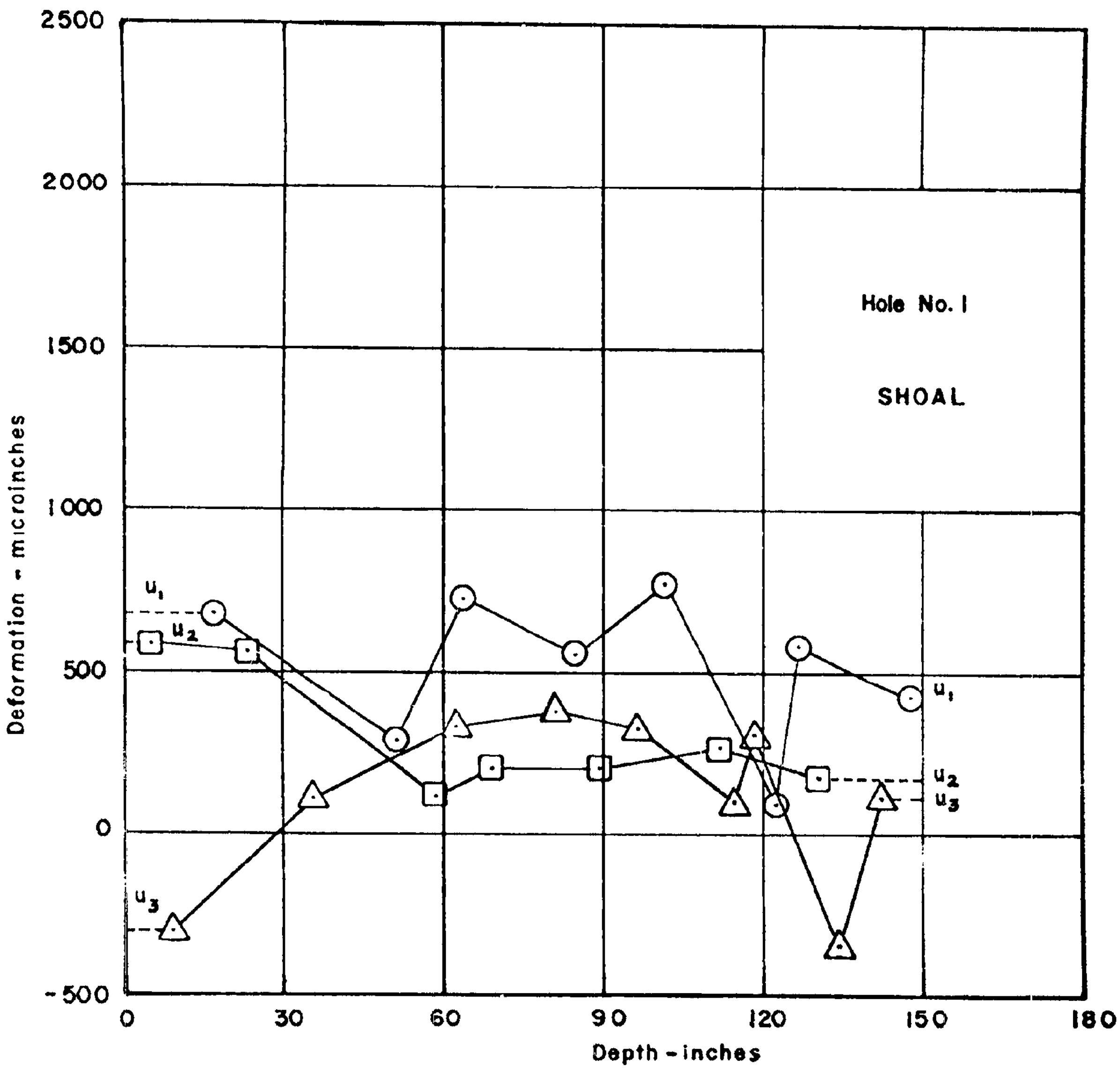

Fig. 4 - BOREHOLE DEFORMATION DATA HOLE NO. 1

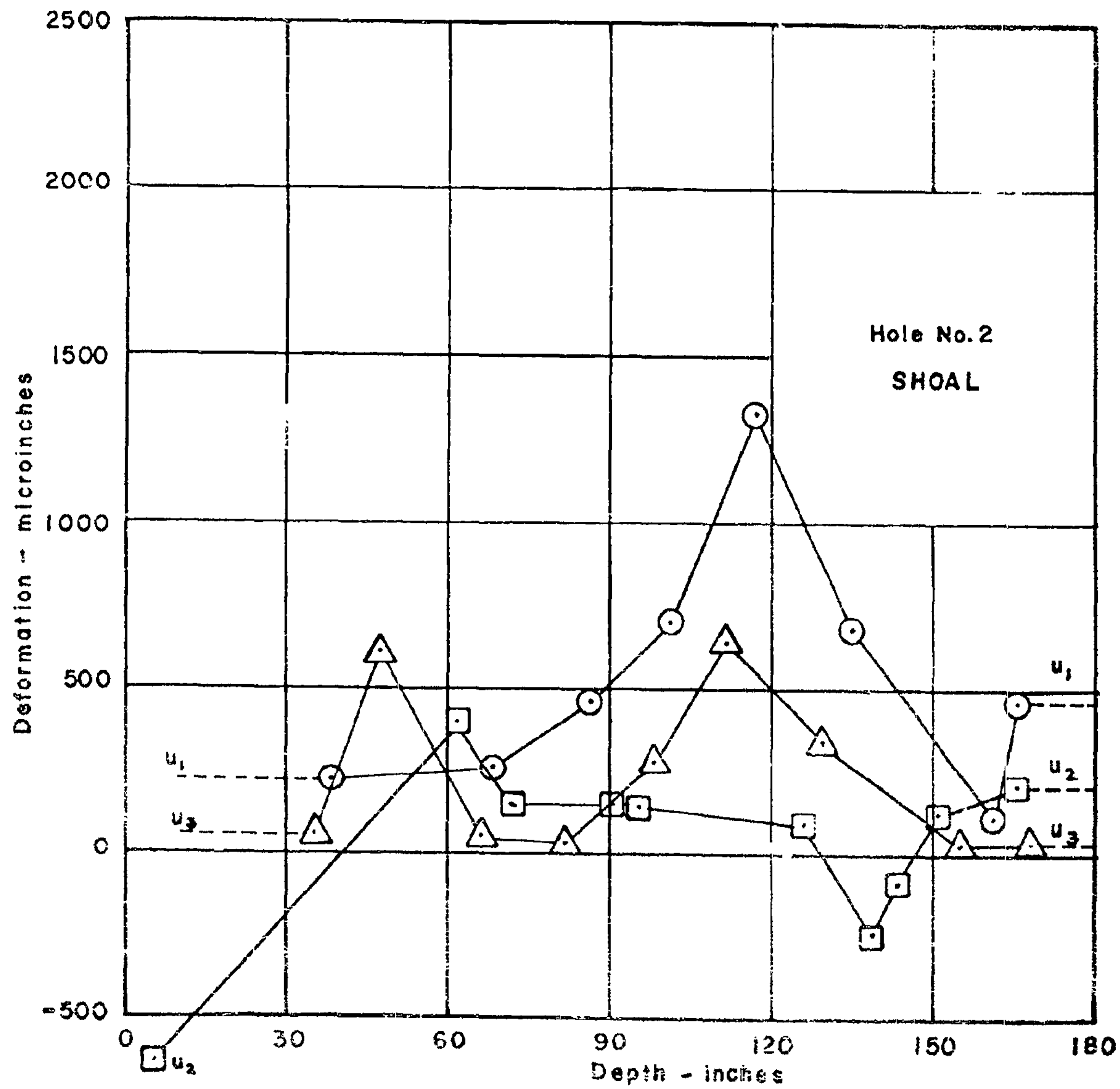

Fig. 5 - BOREHOLE DEFORMATION DATA HOLE NO. 2

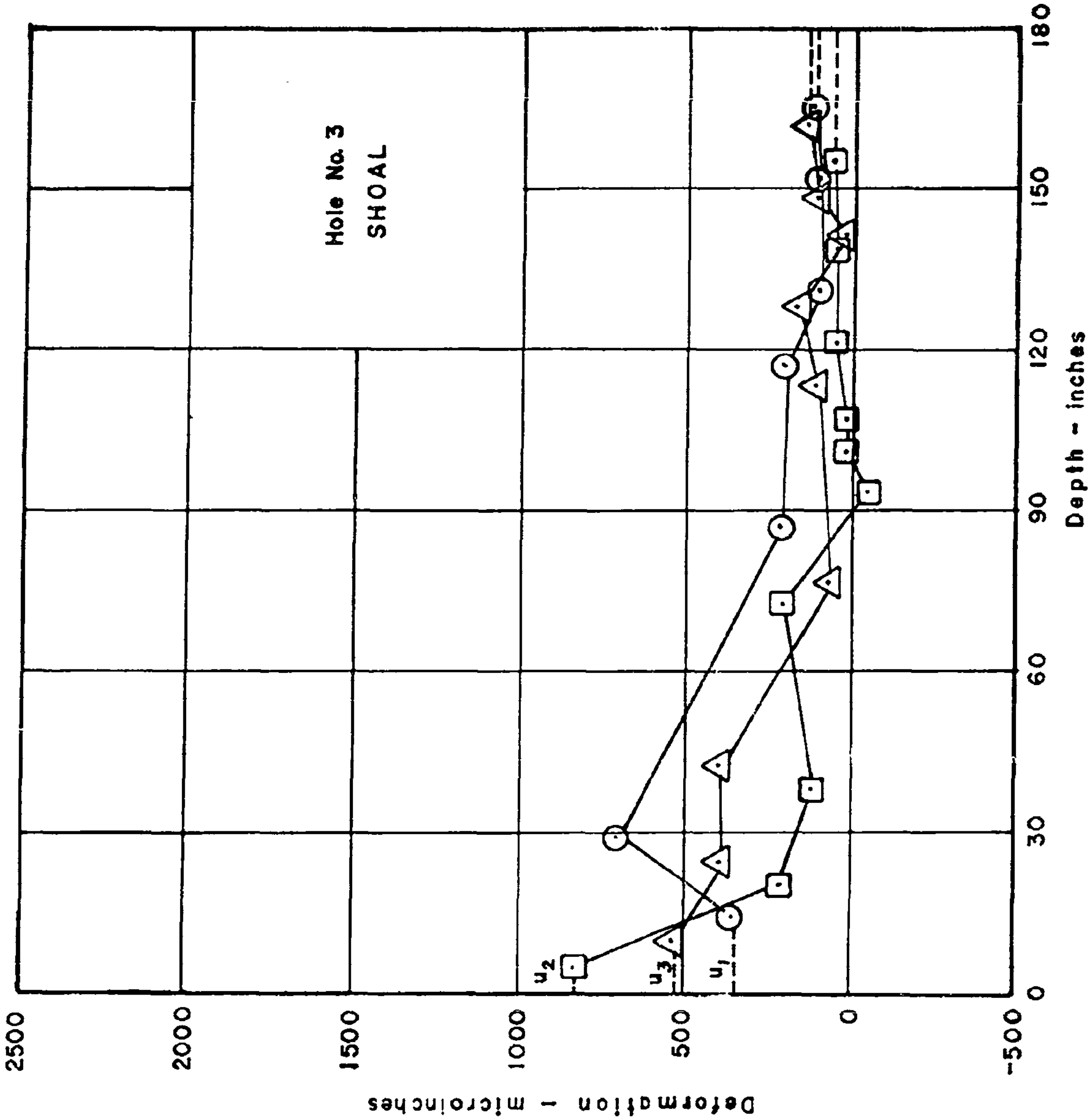

Fig. 6 - BOREHOLE DEFORMATION DATA Hole 3

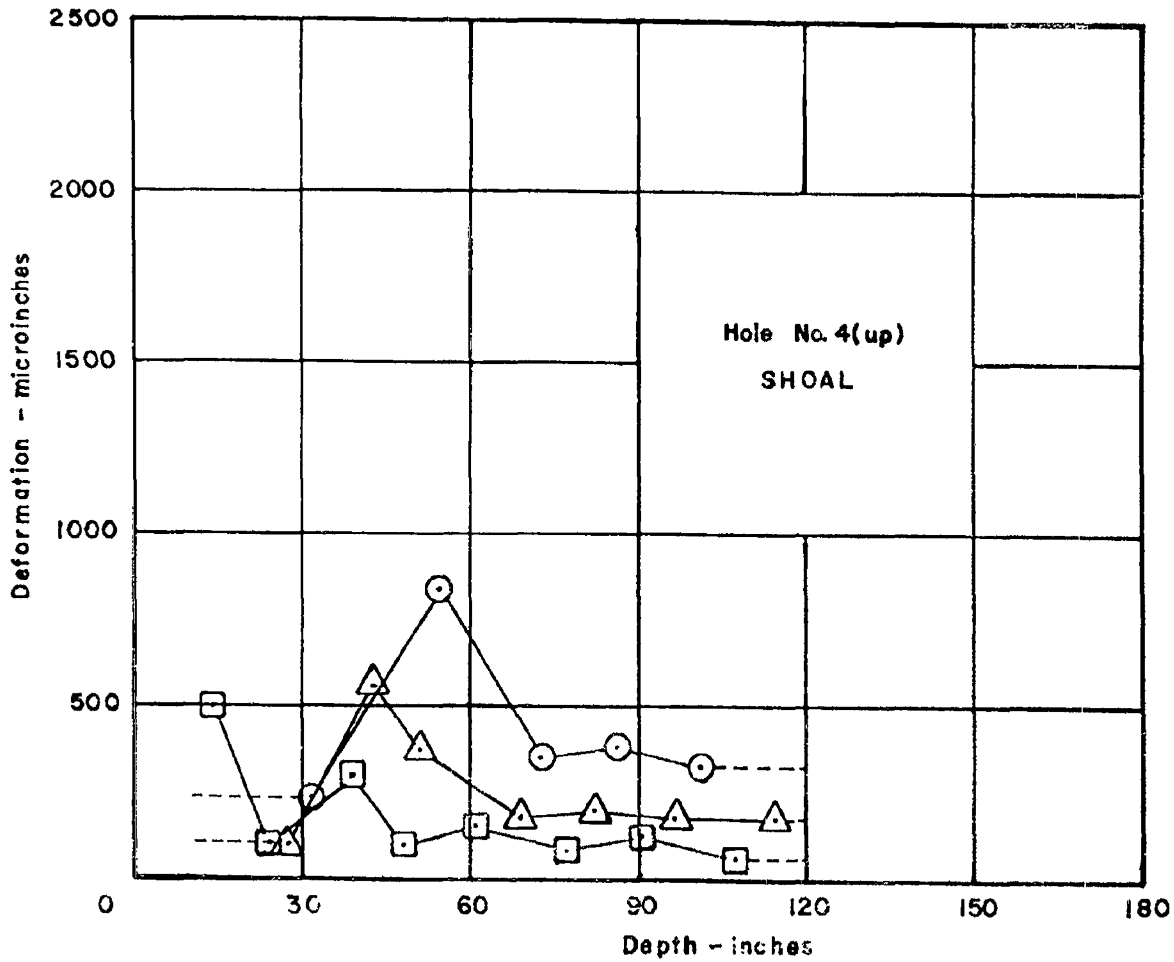

Fig. 7 -BOREHOLE DEFORMATION DATA HOLE NO. 4(up)

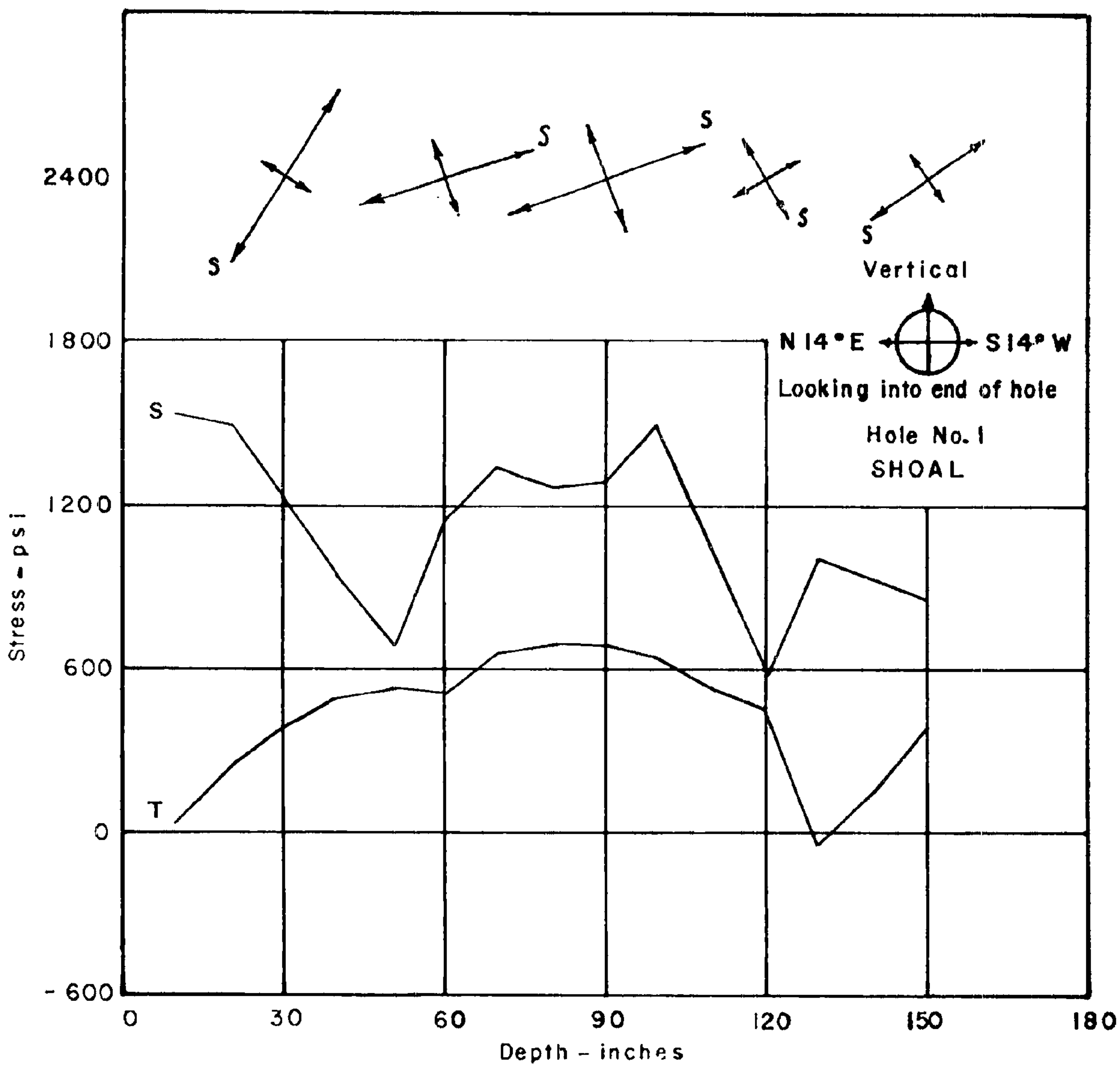

Fig. 8 - Stress vs. Distance from Face Hole No. 1

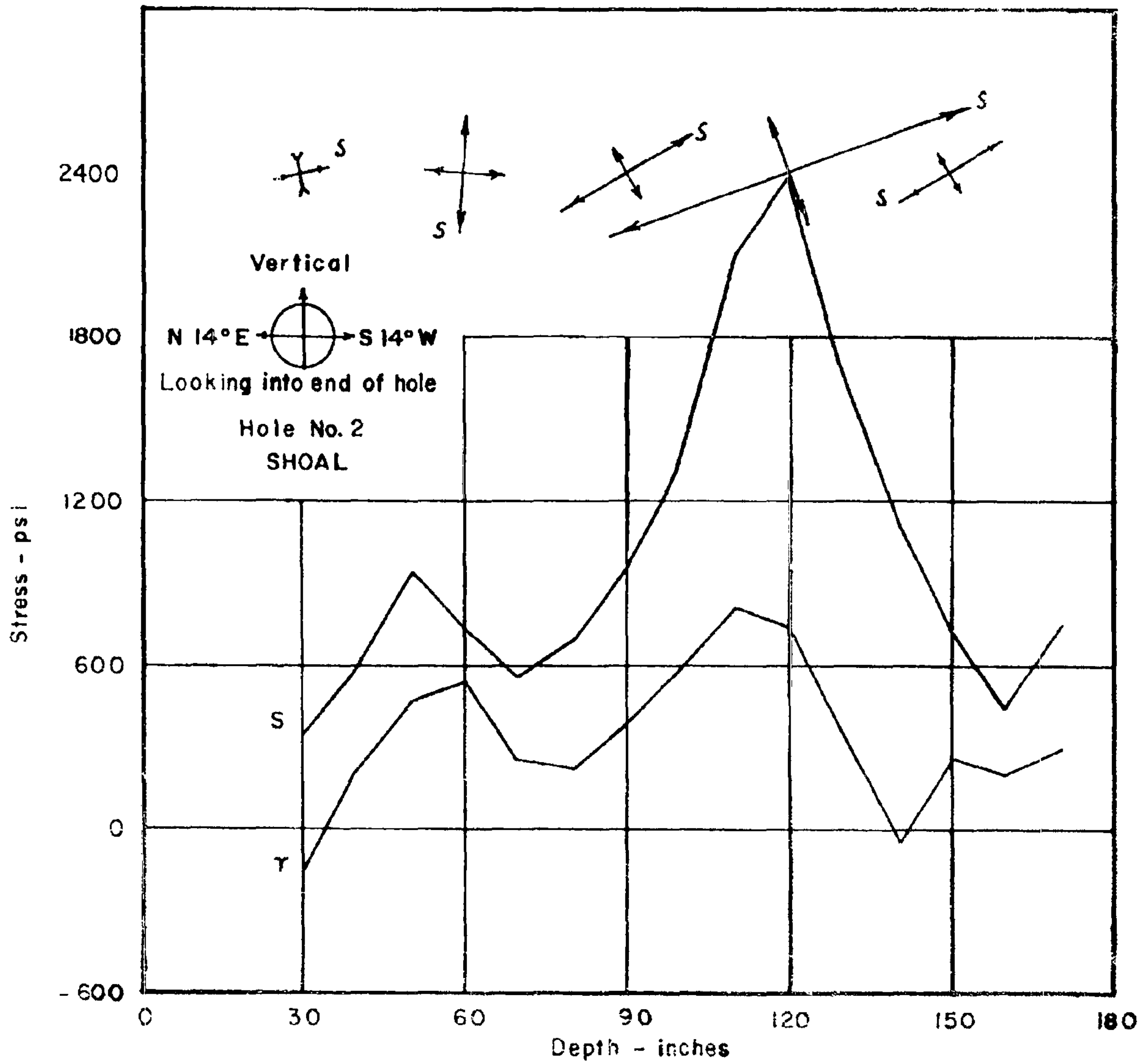

Fig. 9 - Stress vs. Distance from Face Hole No. 2

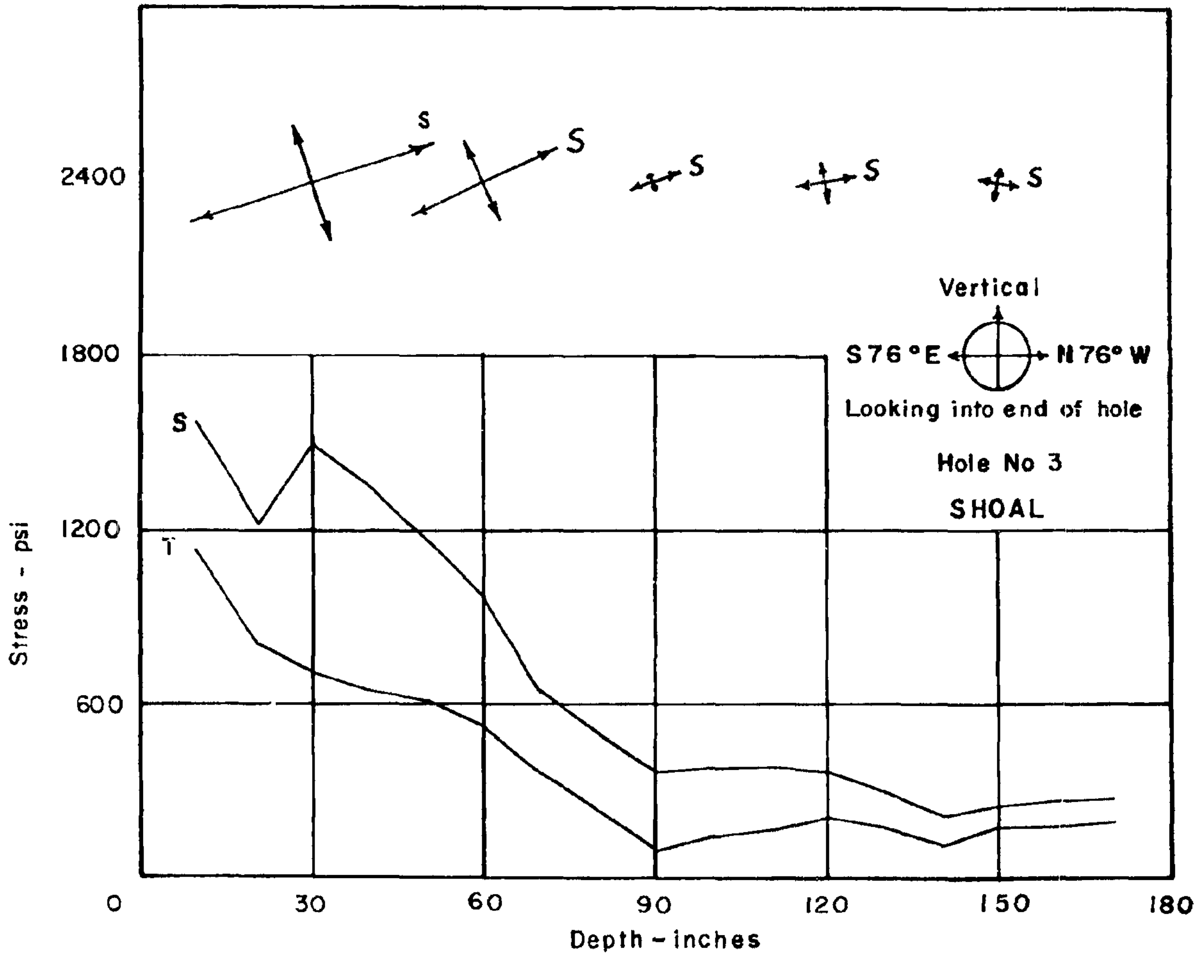

Fig. 10– Stress vs. Distance from Face Hole 3

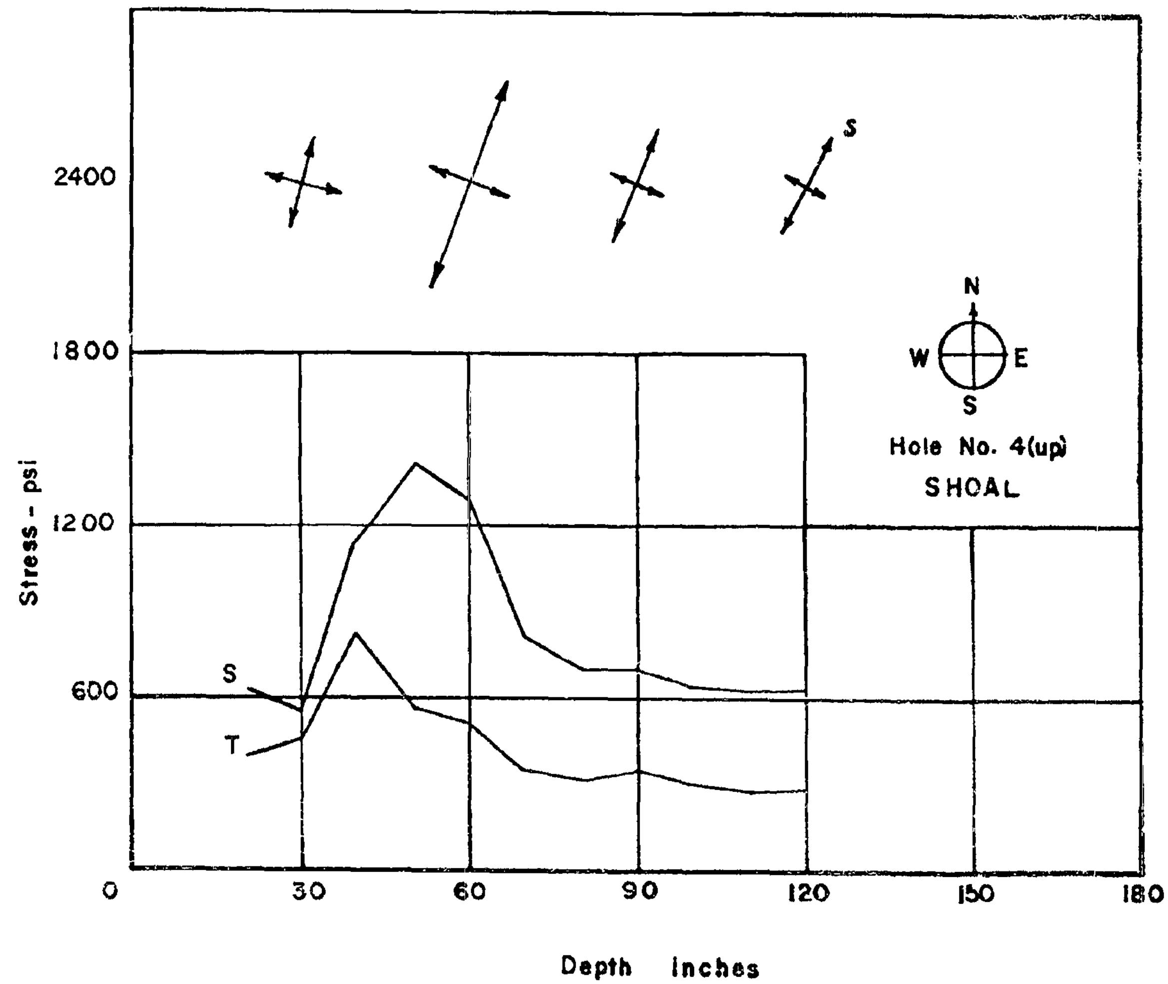

Fig. 11 Stress vs. Distance from Face Hole 4 (up)

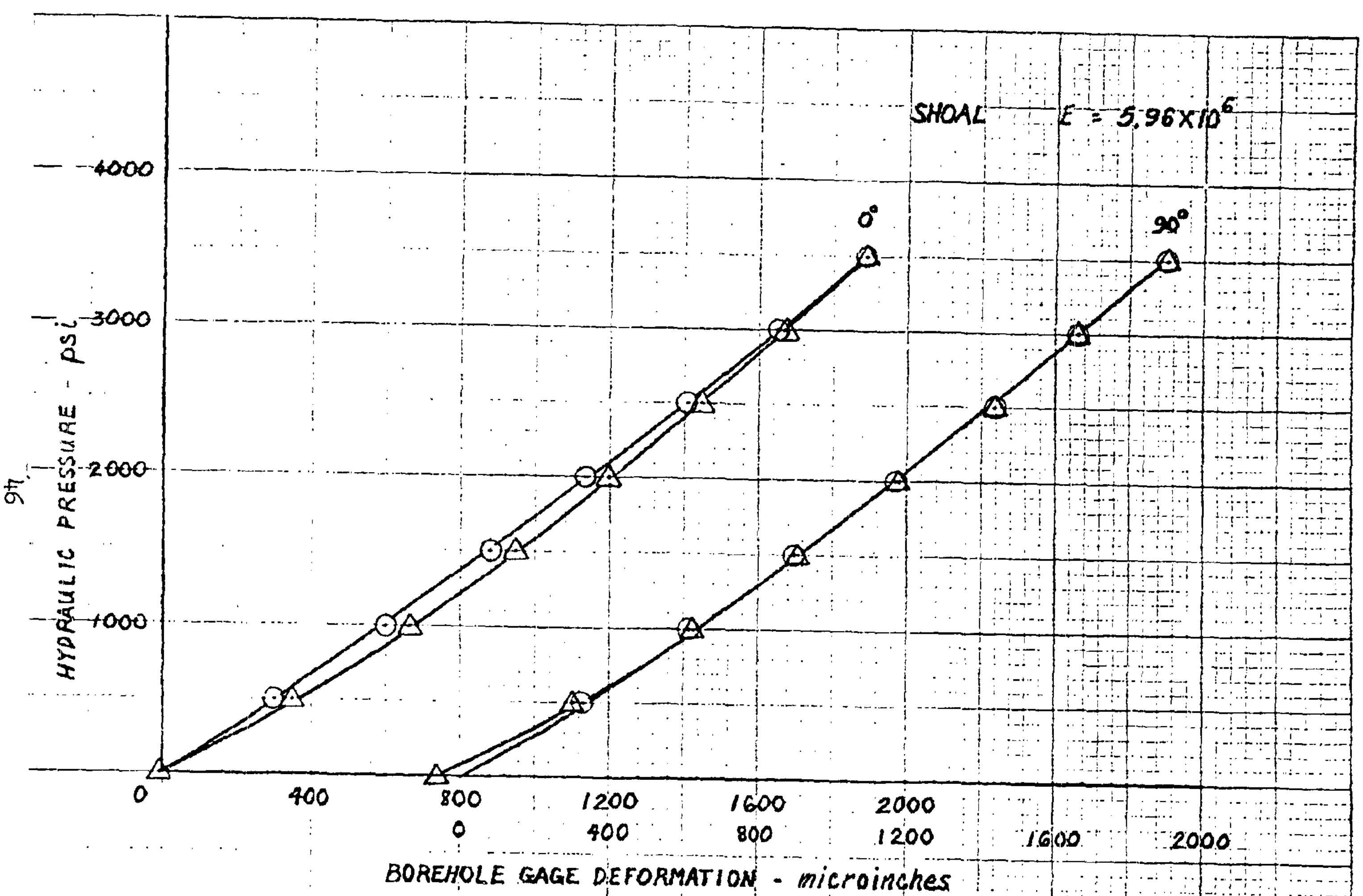

Fig. 12 - DATA FOR MODULUS OF ELASTICITY AT 150 INCH DEPTH - HOLE 3

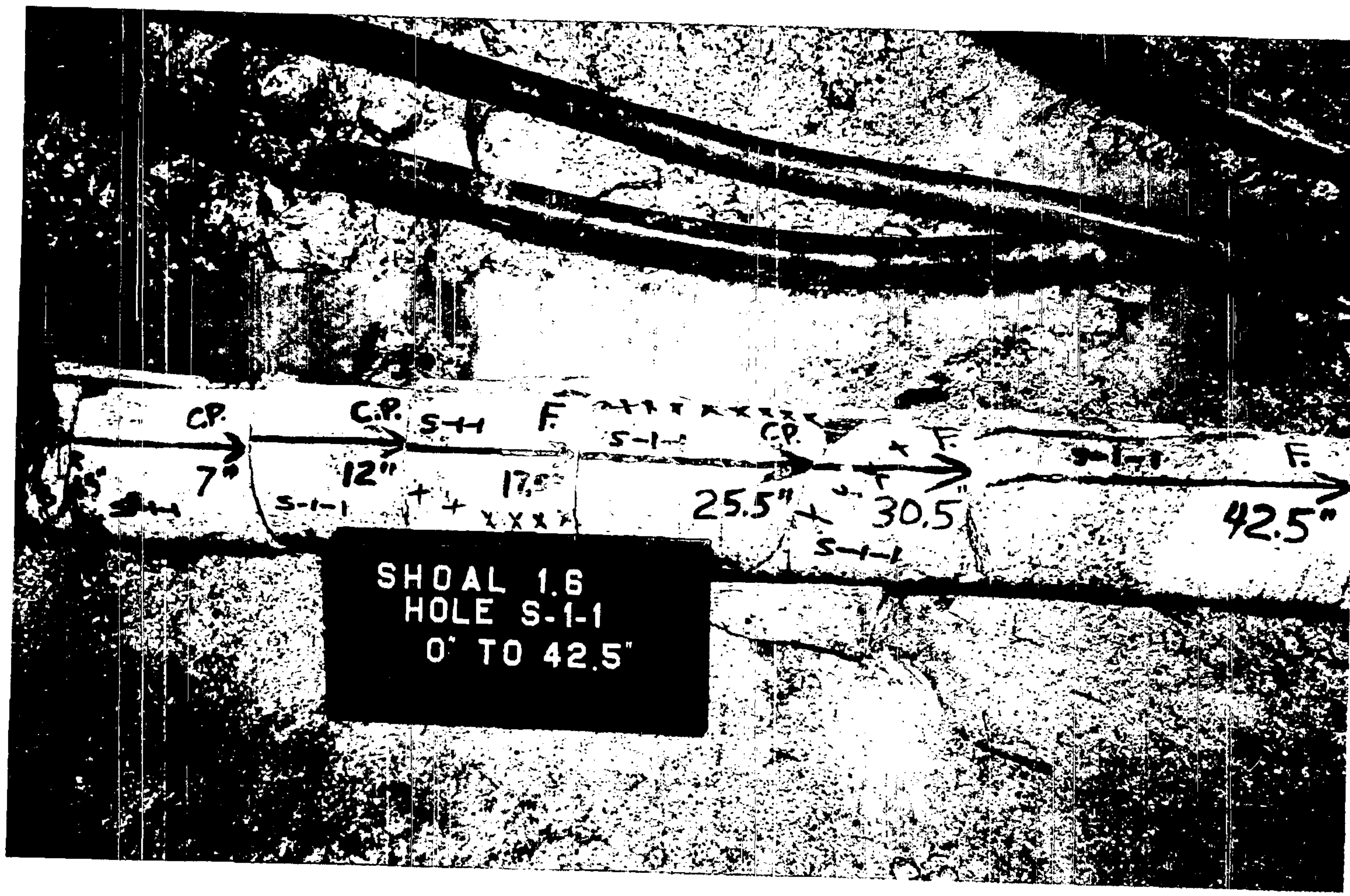

Photo. 1a Hole 1. Core Interval
0 to 42.5 inches

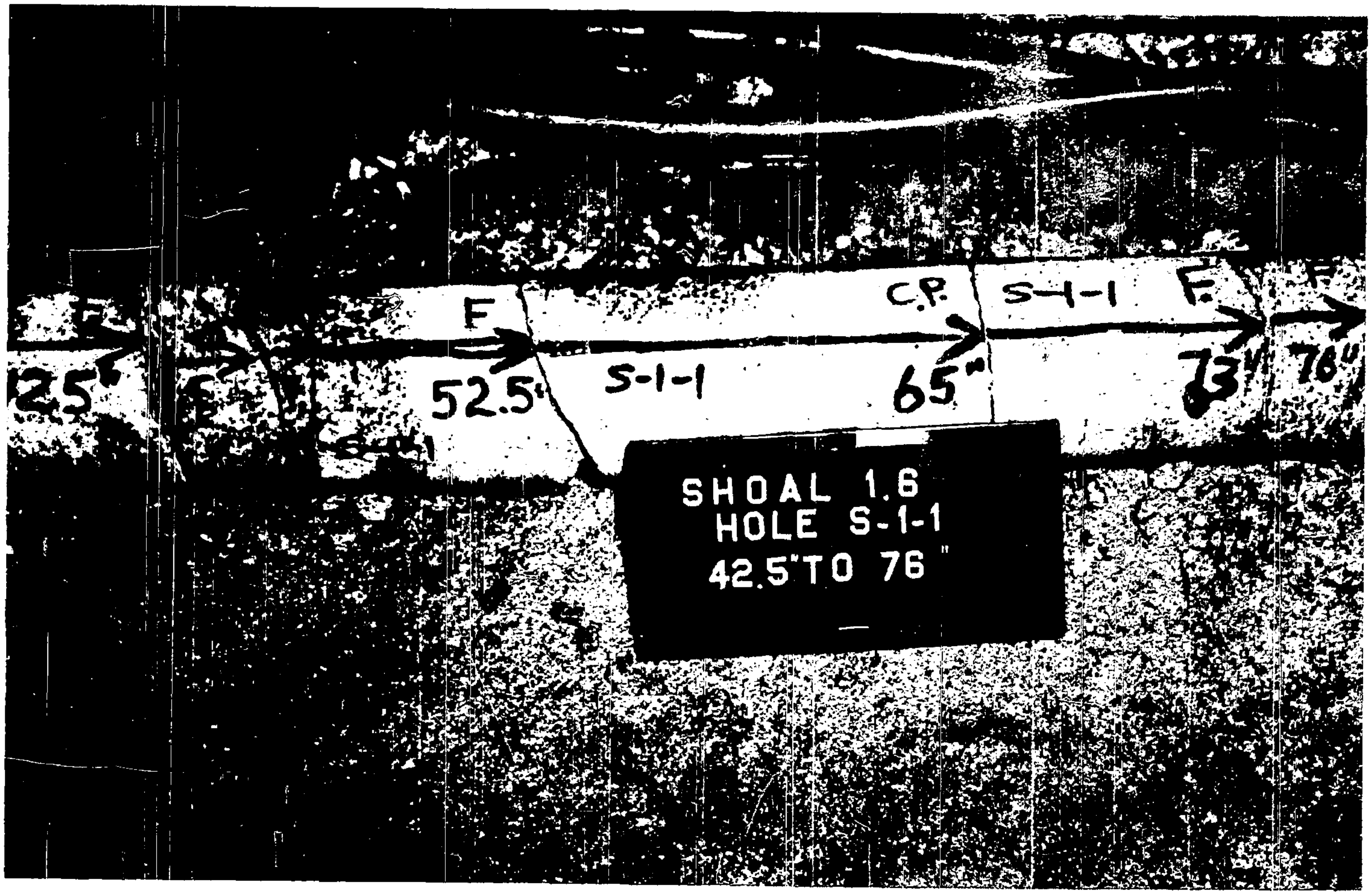

Photo. 1b Hole 1. Core Interval
42.5 to 76 inches

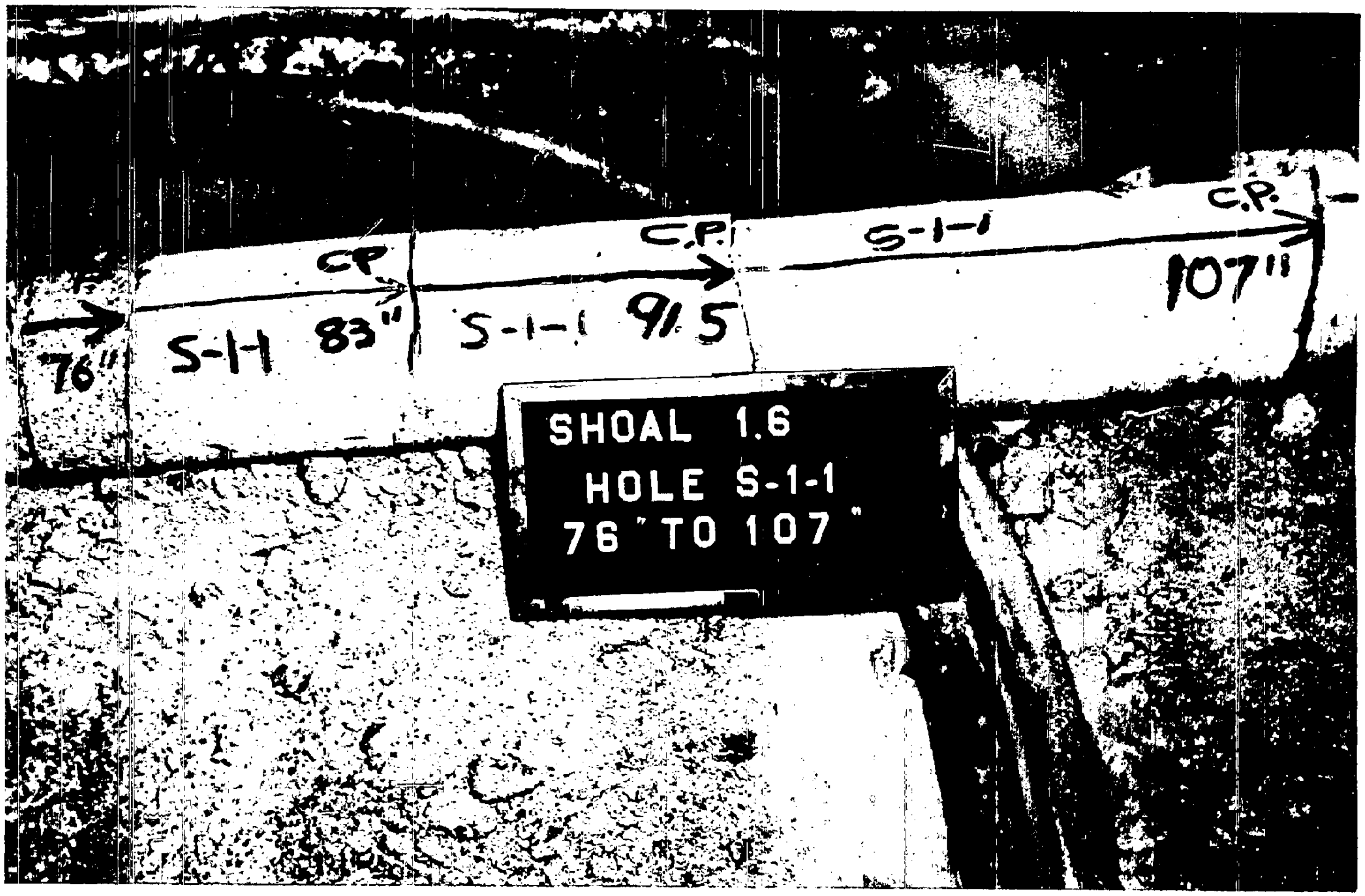

Photo. 1c Hole 1. Core Interval
76 to 107 inches

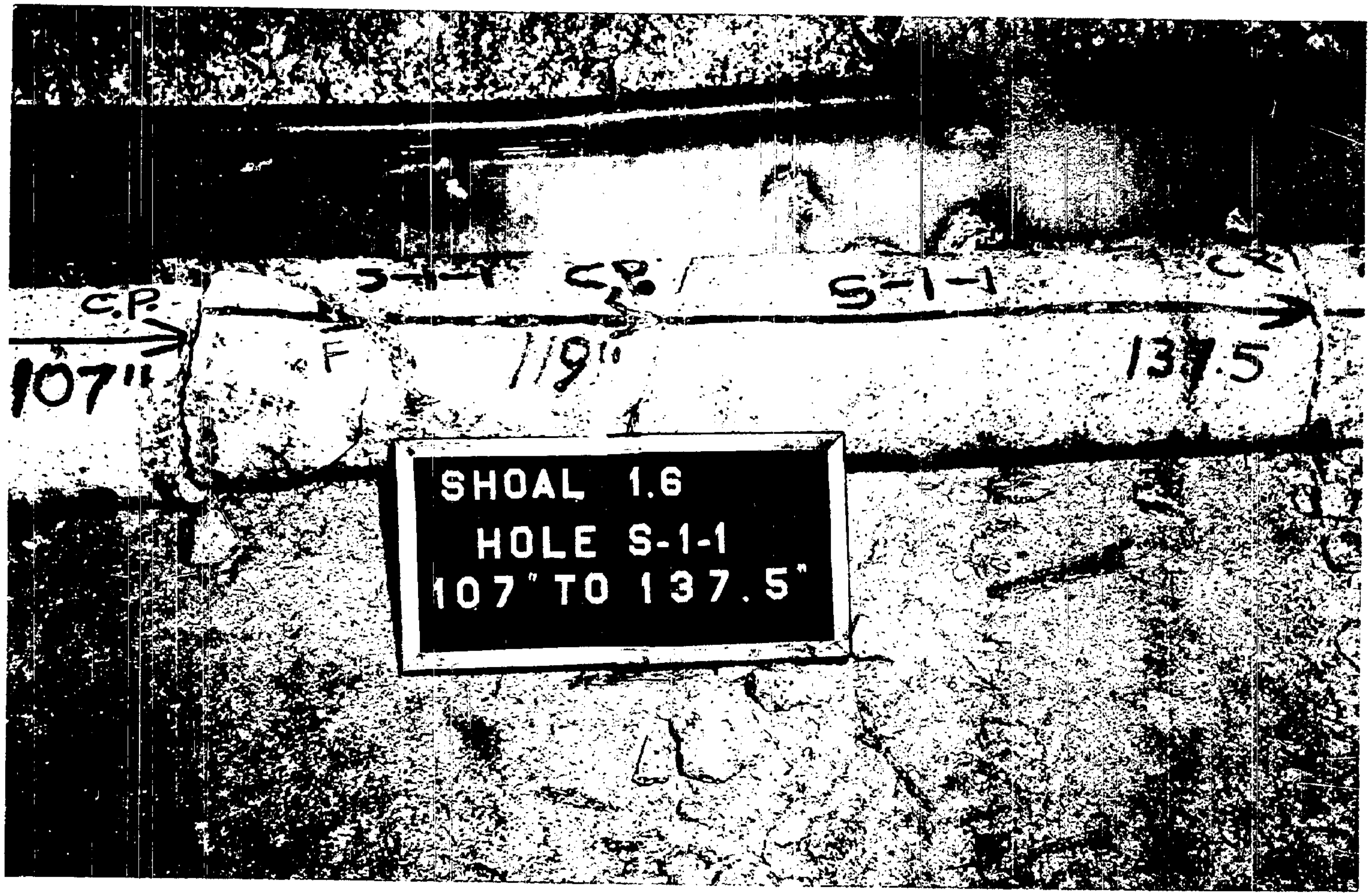

Photo. 1d Hole 1. Core Interval
107 to 137.5 inches

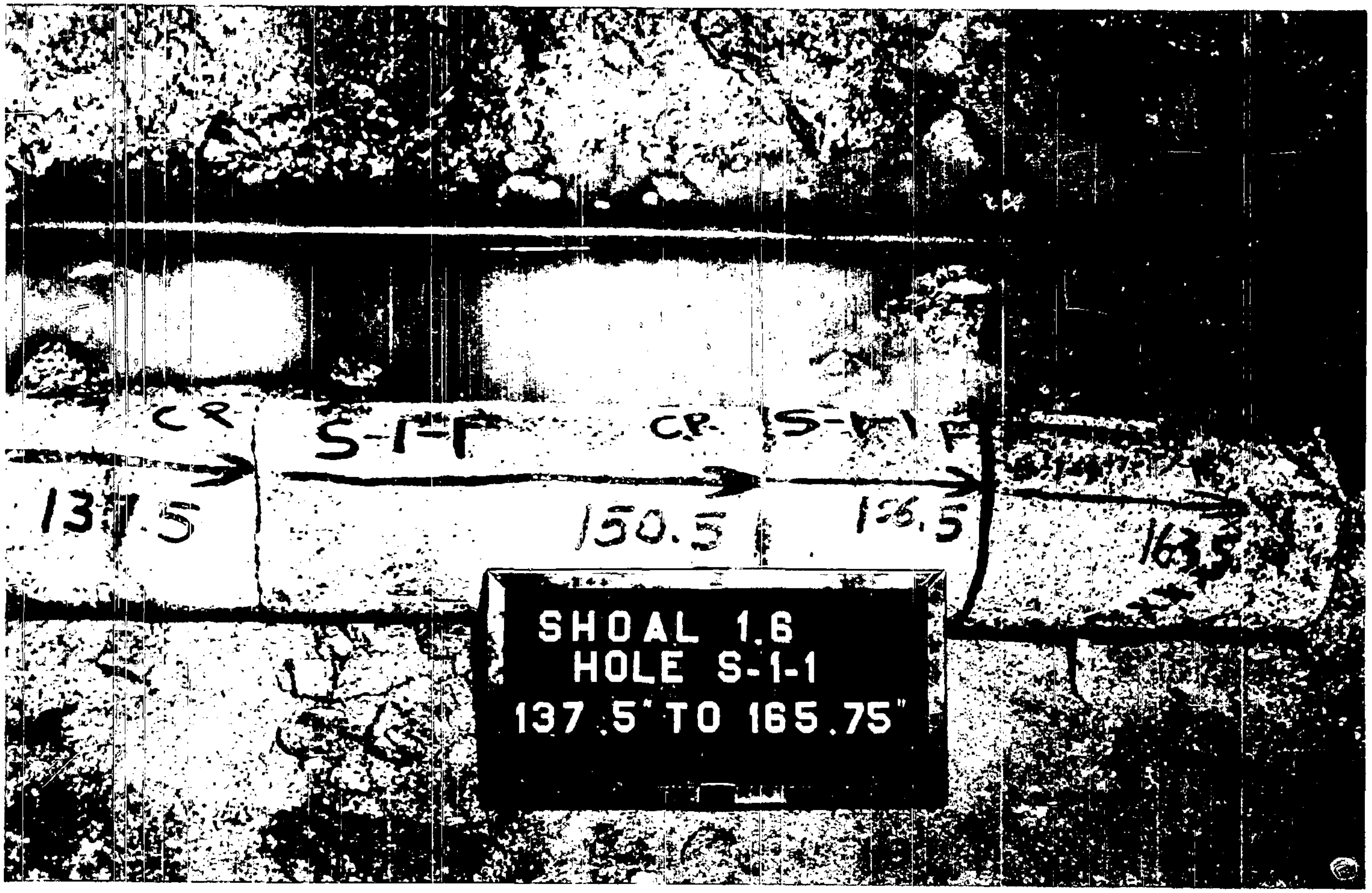

Photo. 1e Hole 1. Core Interval
137.5 to 165.75 inches

Photo. 2a Hole 2. Core Interval
0 to 42 inches

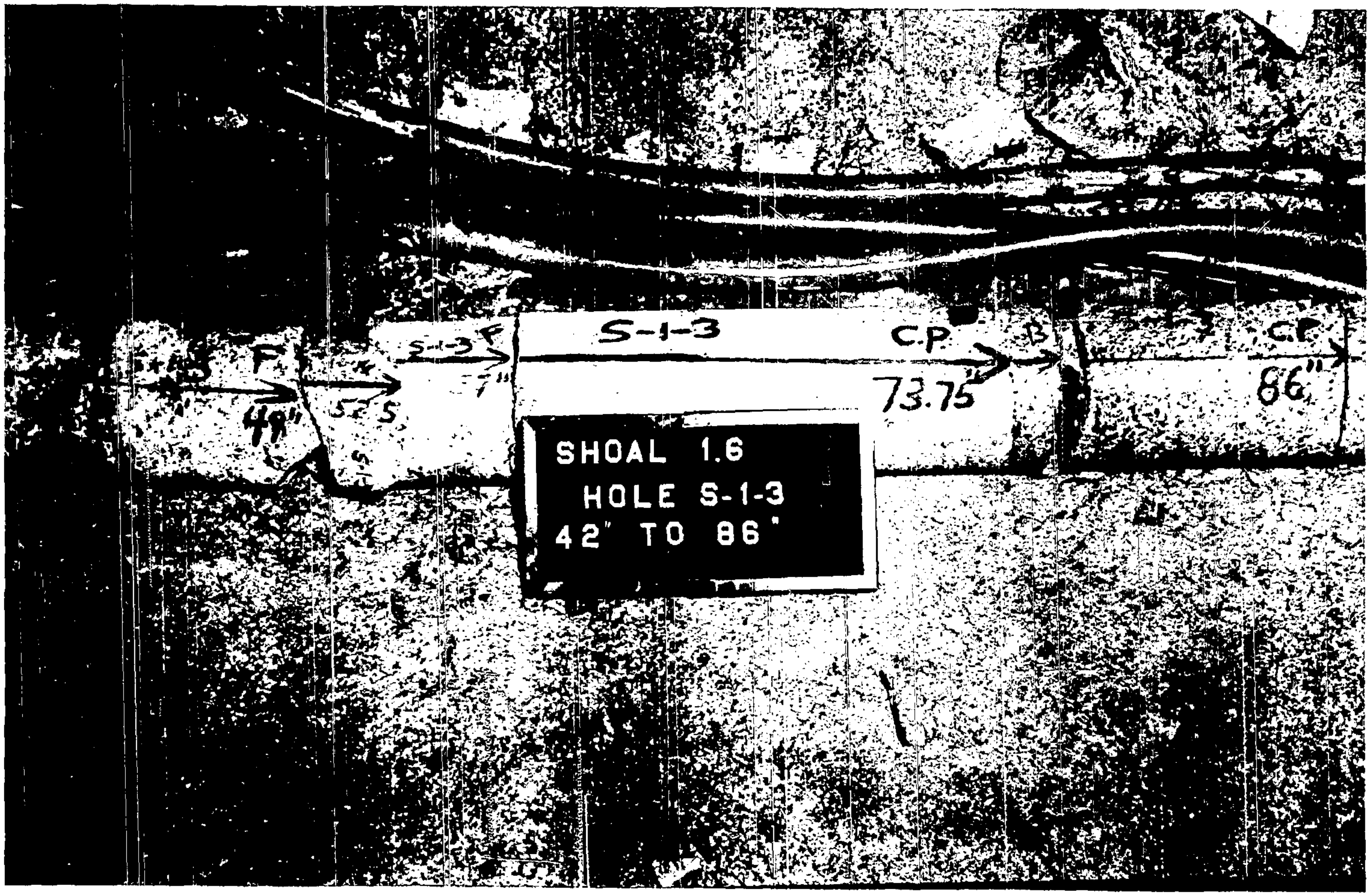

Photo. 2b Hole 2. Core Interval
42 to 86 inches

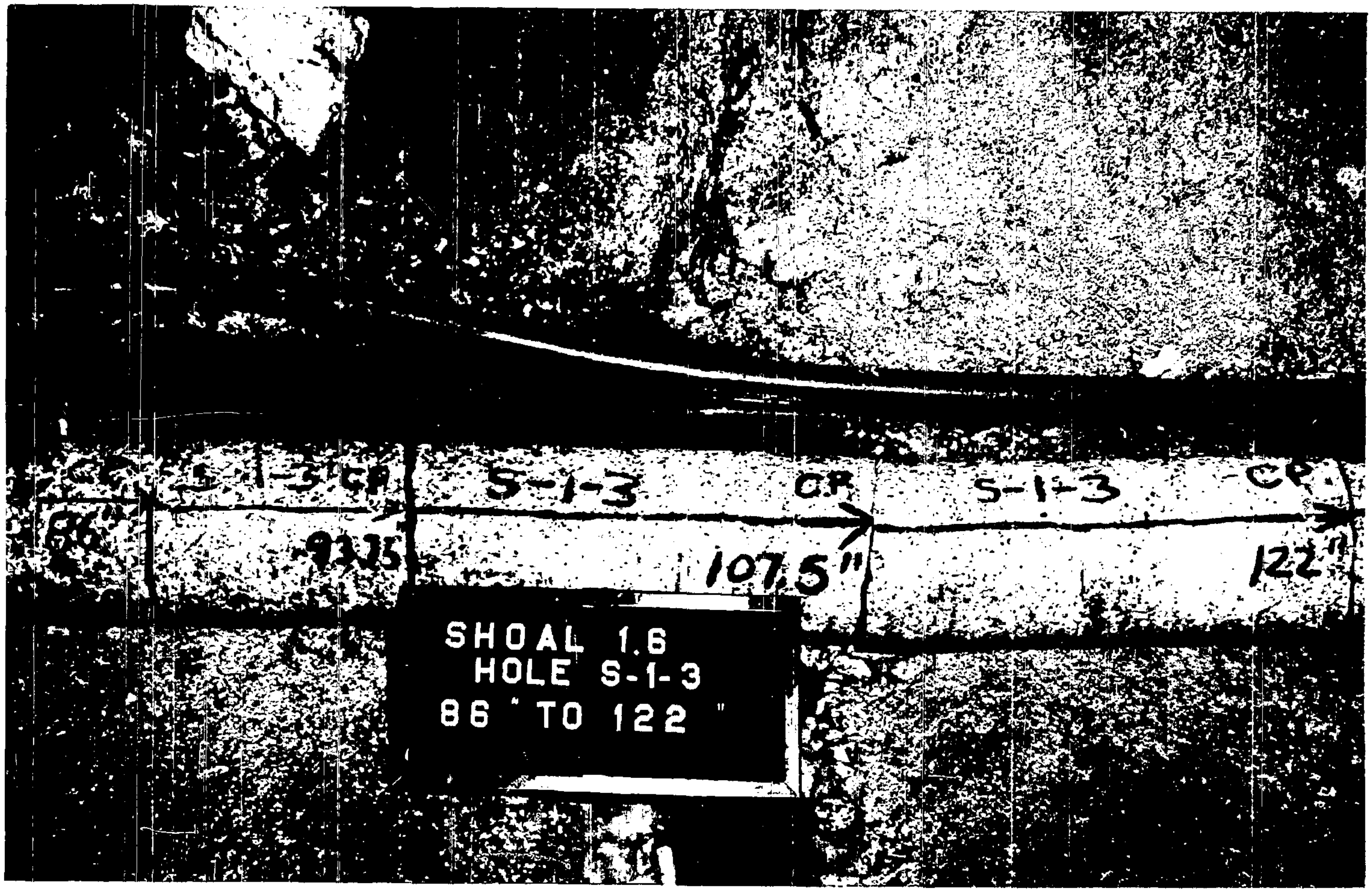

Photo. 2c Hole 2. Core Interval
86 to 122 inches

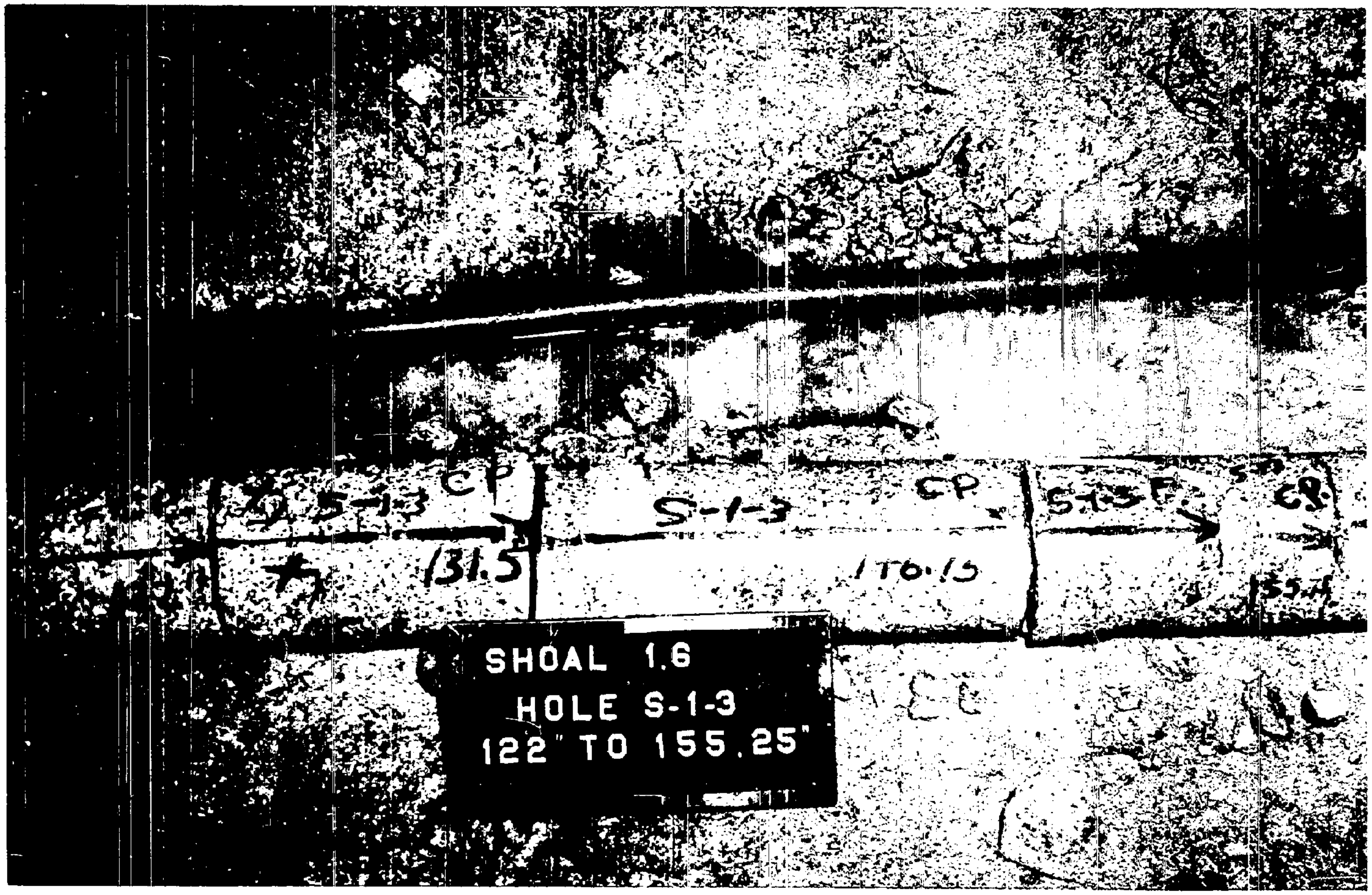

Photo. 2d Hole 2. Core Interval
122 to 155.25 inches

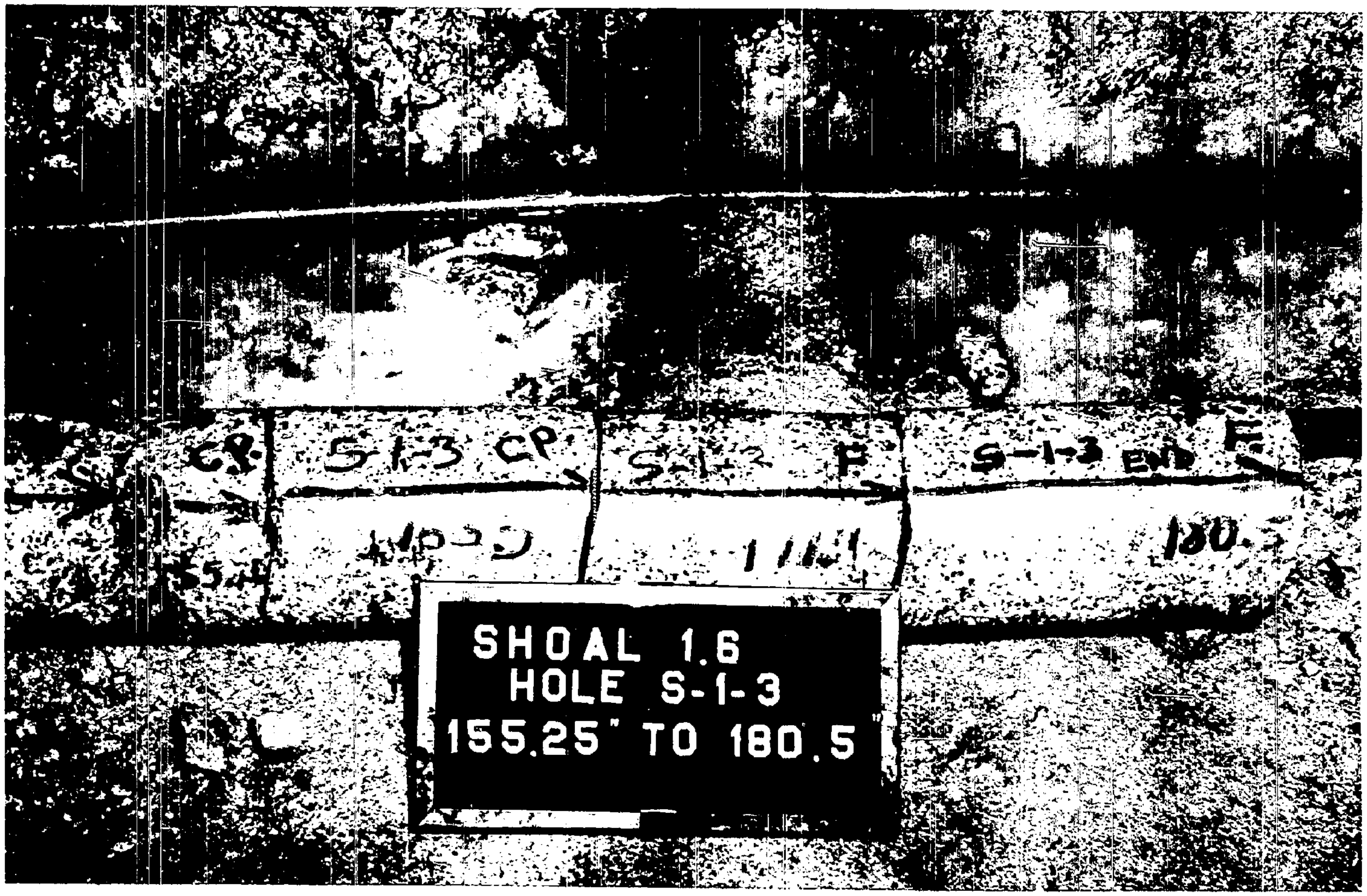

Photo. 2e Hole 2. Core Interval
155.25 to 180.5 inches

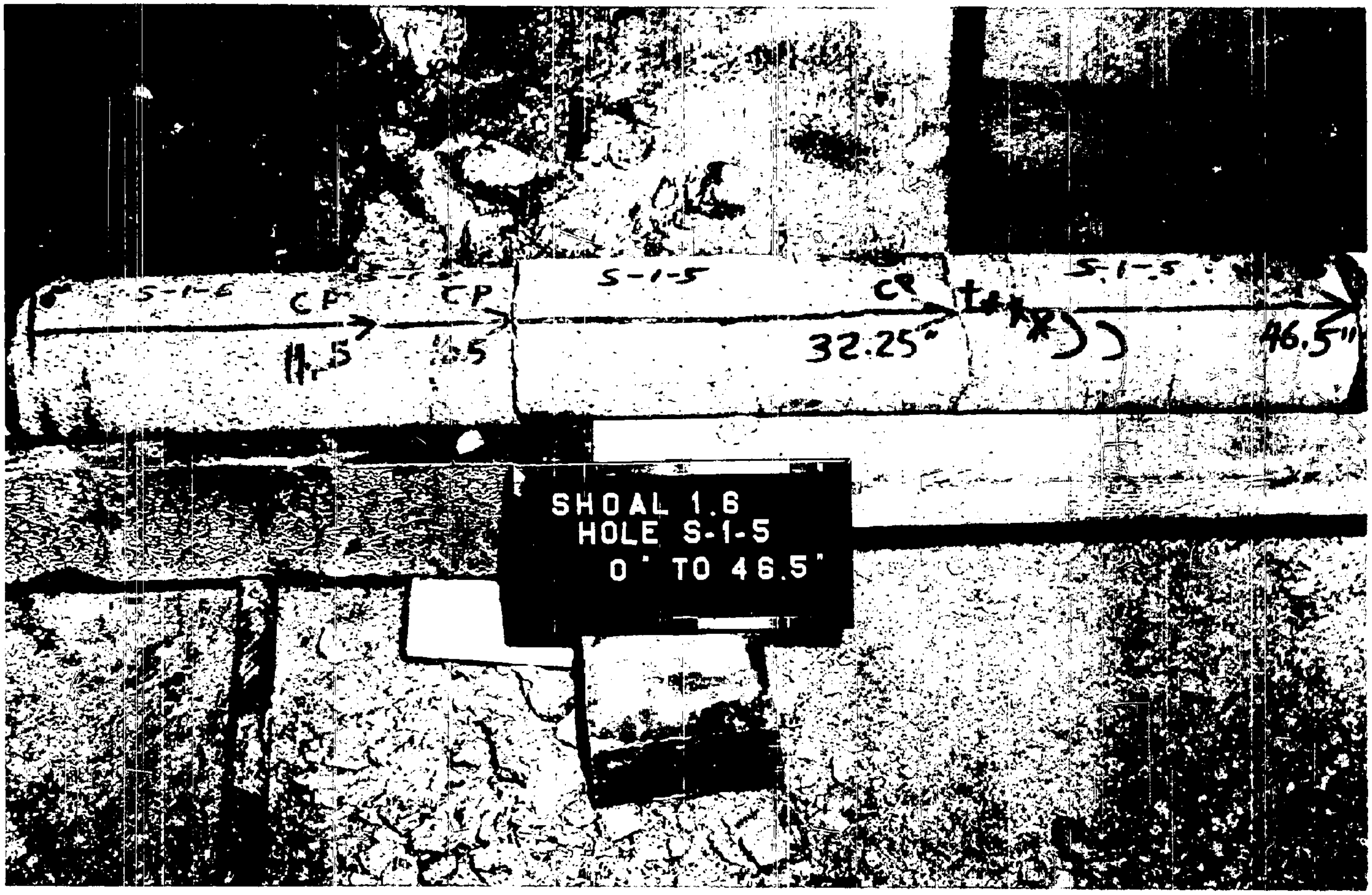

Photo. 3a Hole 3. Core Interval
0 to 46.5

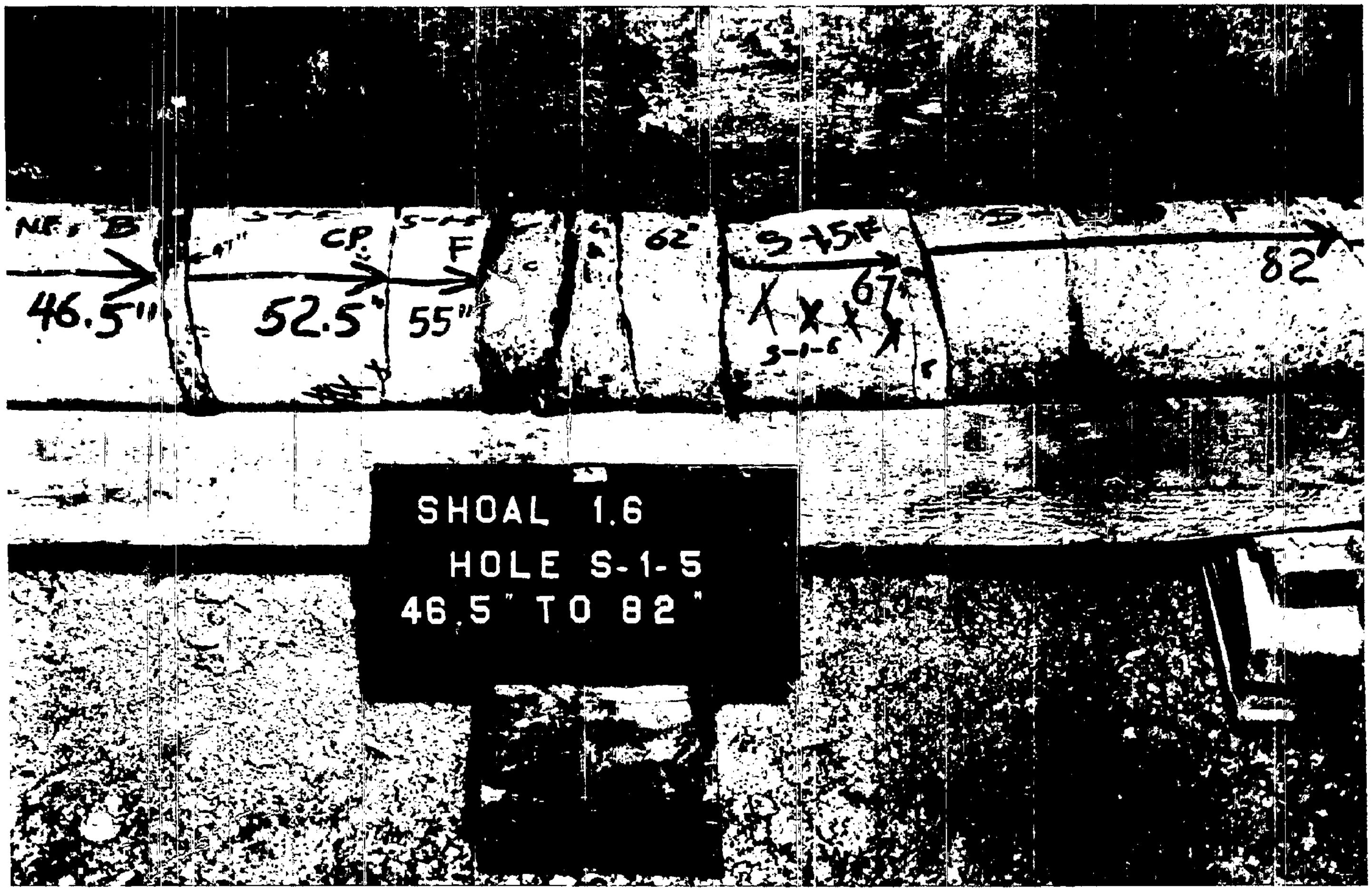

Photo. 3b Hole 3. Core Interval
46.5 to 82 inches

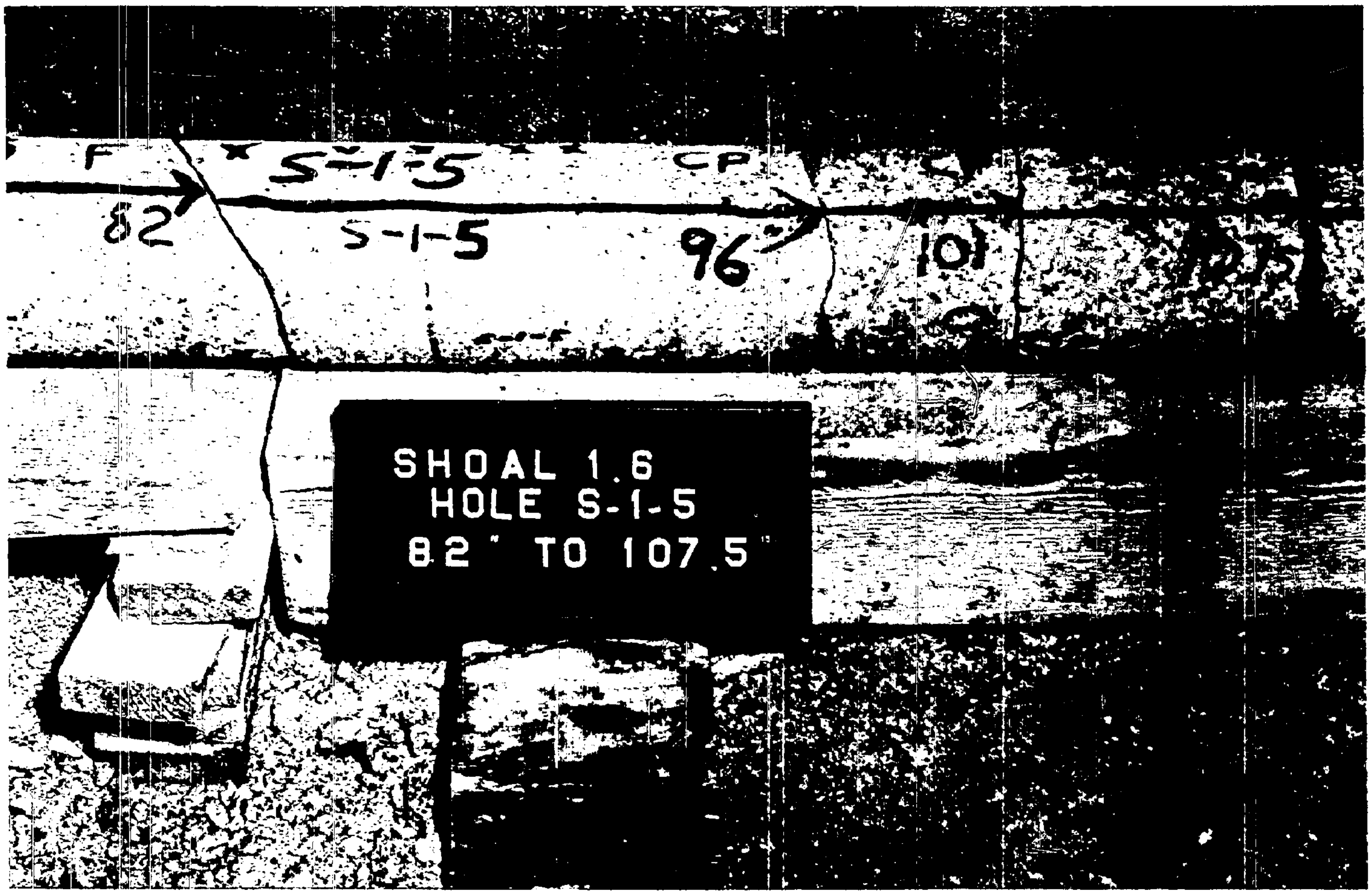

Photo. 3c Hole 3. Core Interval
82 to 107.5 inches

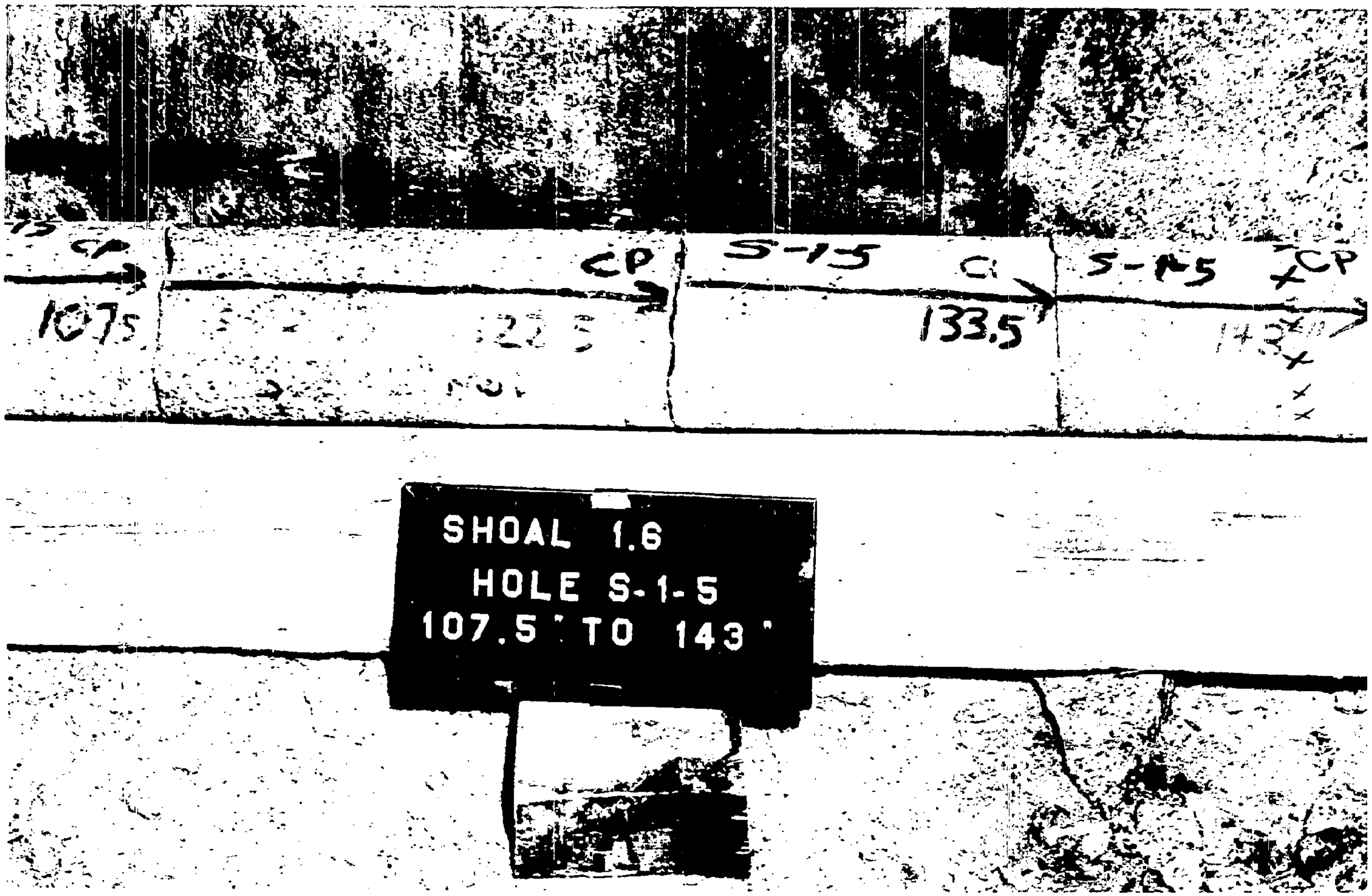

Photo. 3d Hole 3. Core Interval
107.5 to 143 inches

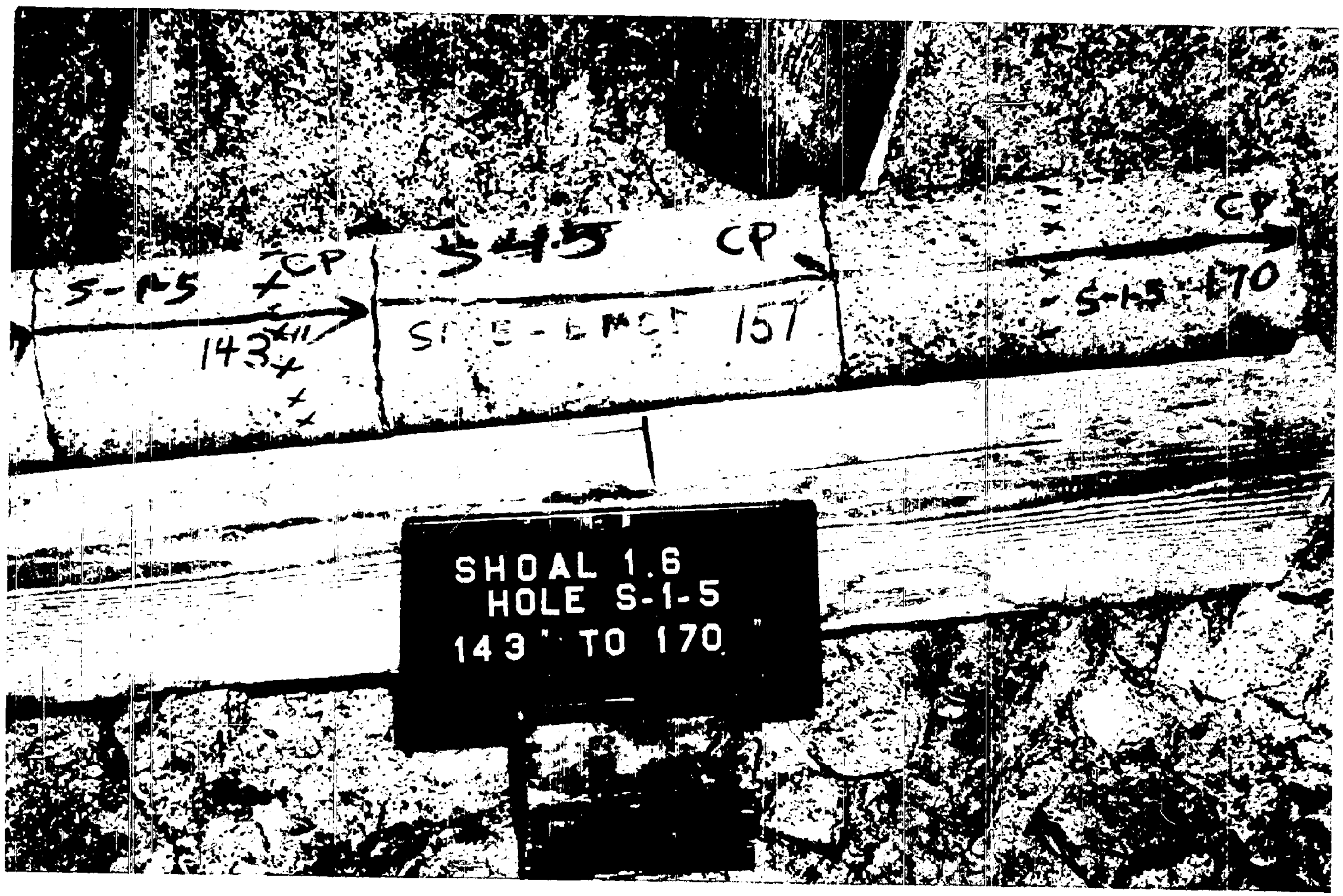

Photo. 3e Hole 3. Core Interval
143 to 170 inches

Photo. 4a(Up) Hole 4. Core Interval
0 to 44.5 inches

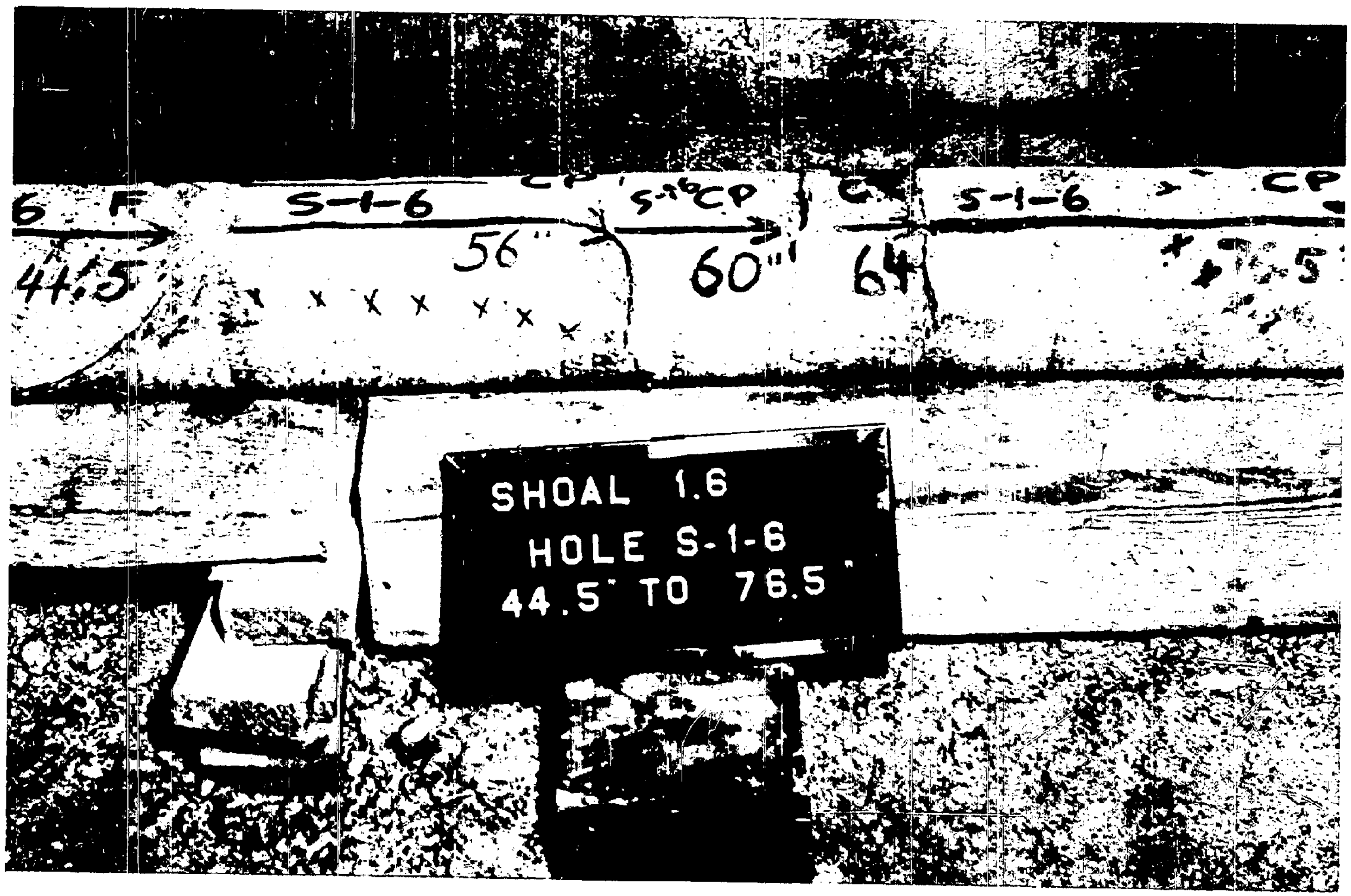

Photo. 4b(Up) Hole 4. Core Interval
44.5 to 76.5 inches

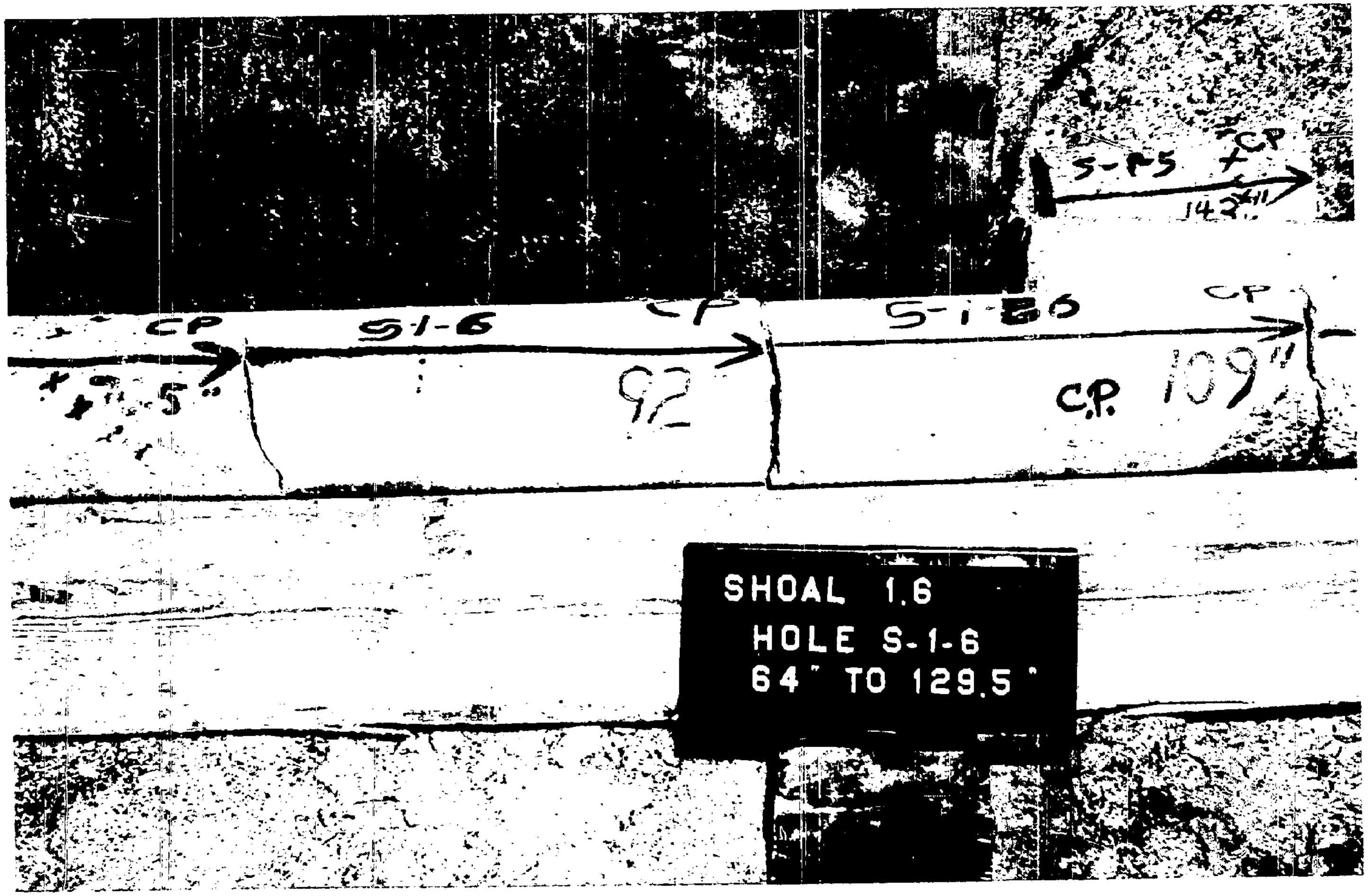

Photo. 4c(Up) Hole 4. Core Interval
76.5 to 109 inches

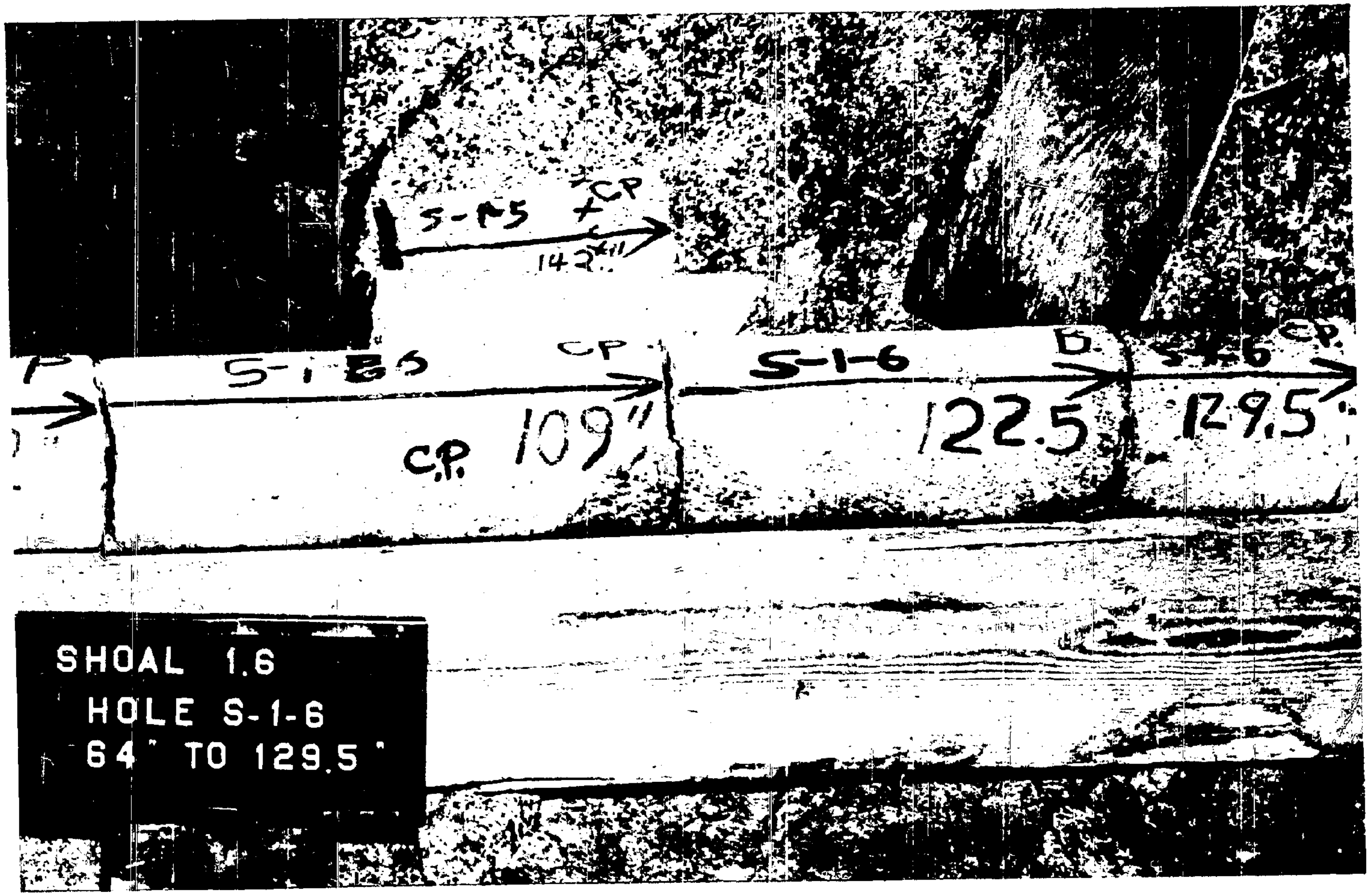

Photo. 4d(Up) Hole 4. Core Interval
109 to 129.5 inches

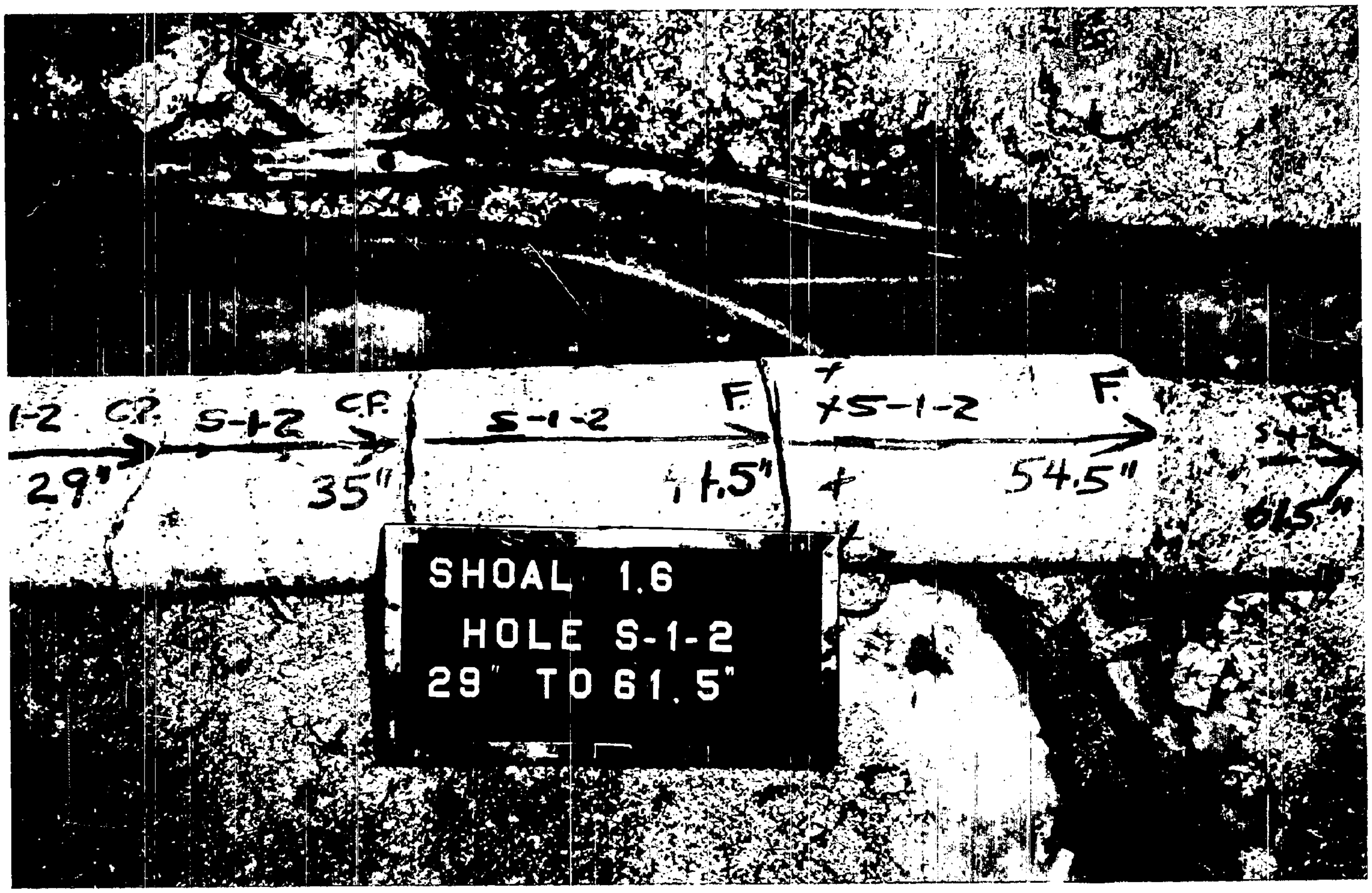

Photo. 5a Hole 5. Core Interval
0 to 29 inches

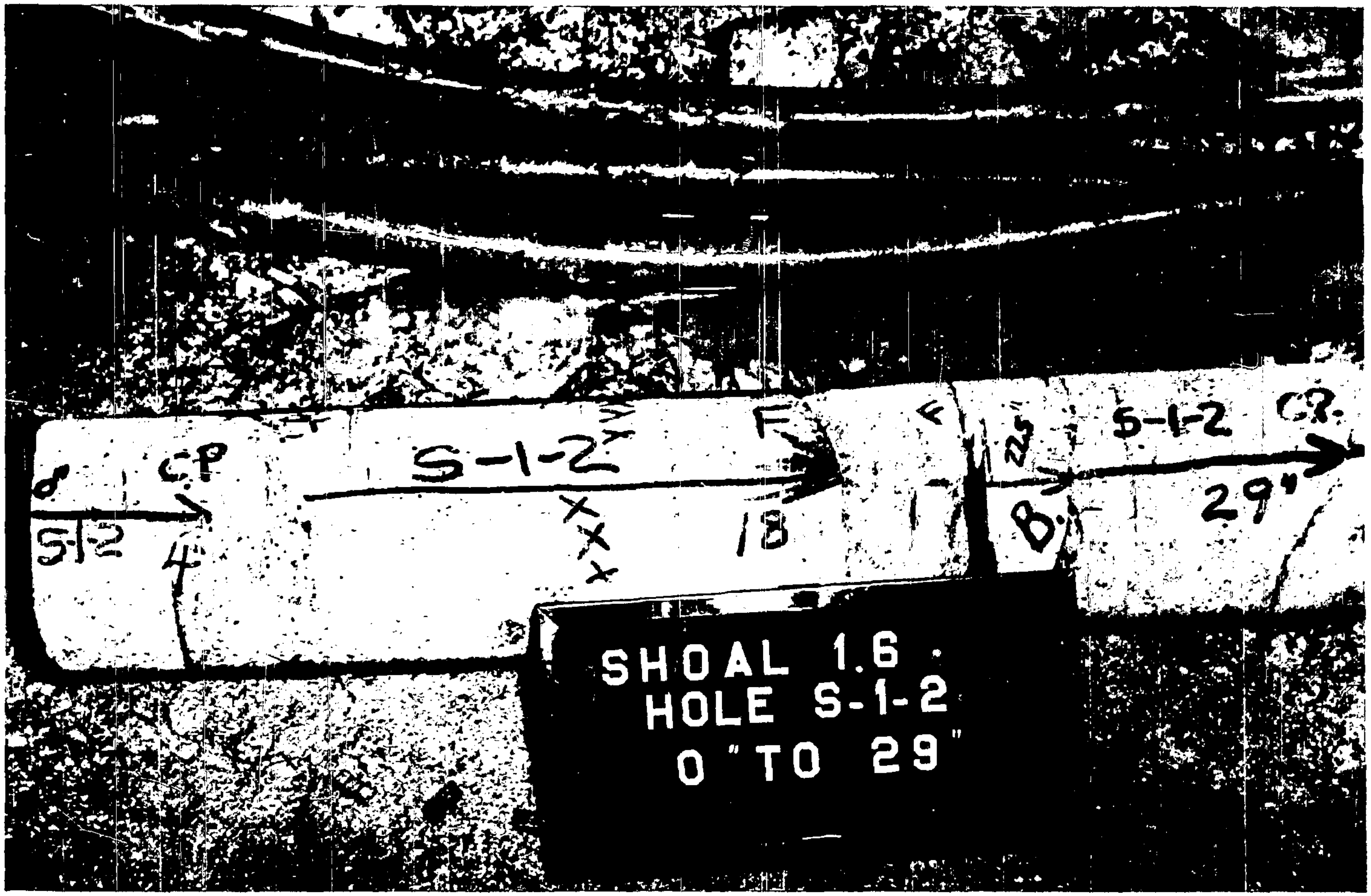

Photo. 5b Hole 5. Core Interval
29 to 61.5 inches

Photo. 6a Hole 6. Core Interval
0 to 41 inches

Photo. 7 Insertion of Borehole Gage

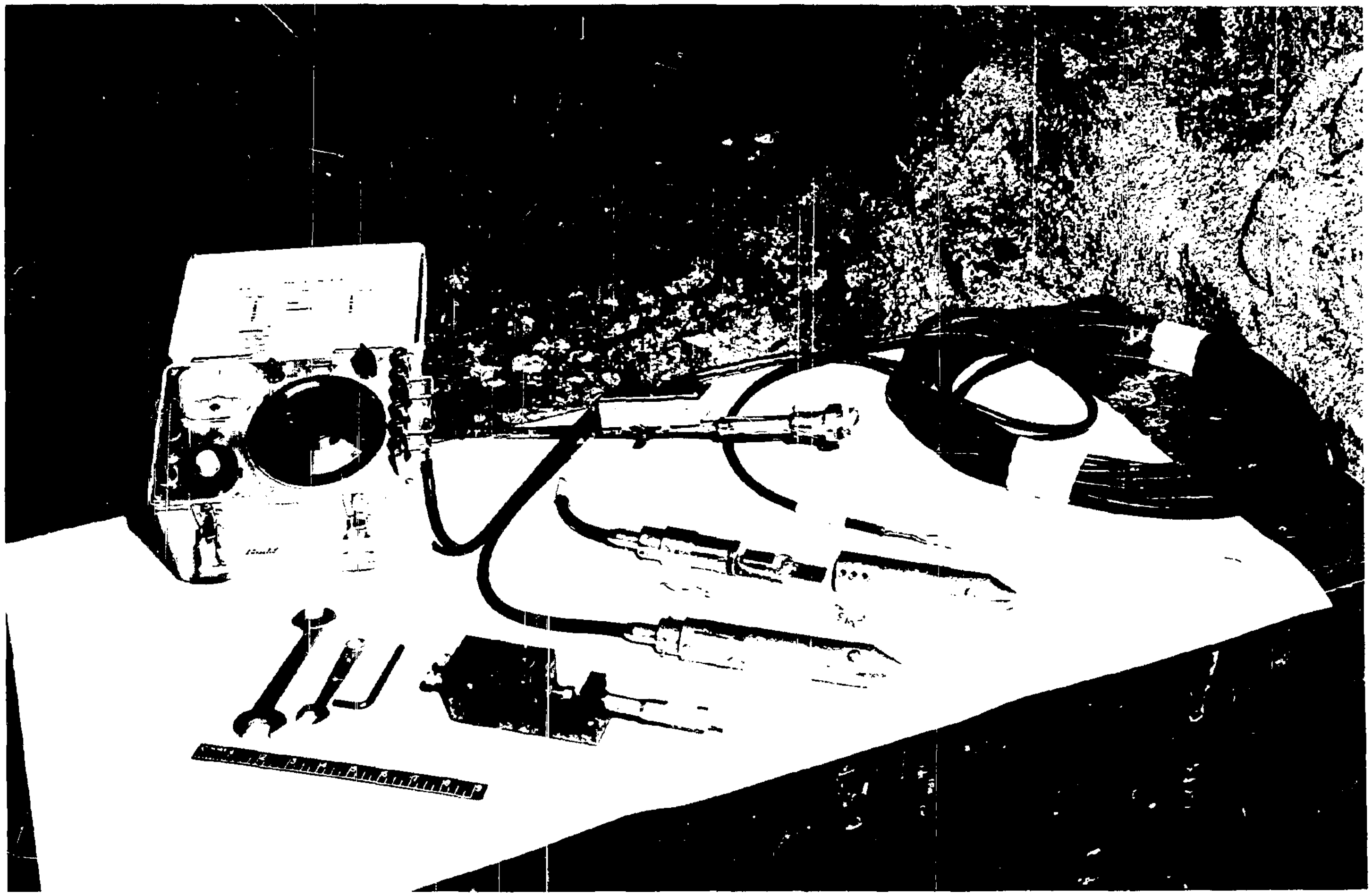

Photo. 8 Borehole Gage, Indicator and Associated Equipment

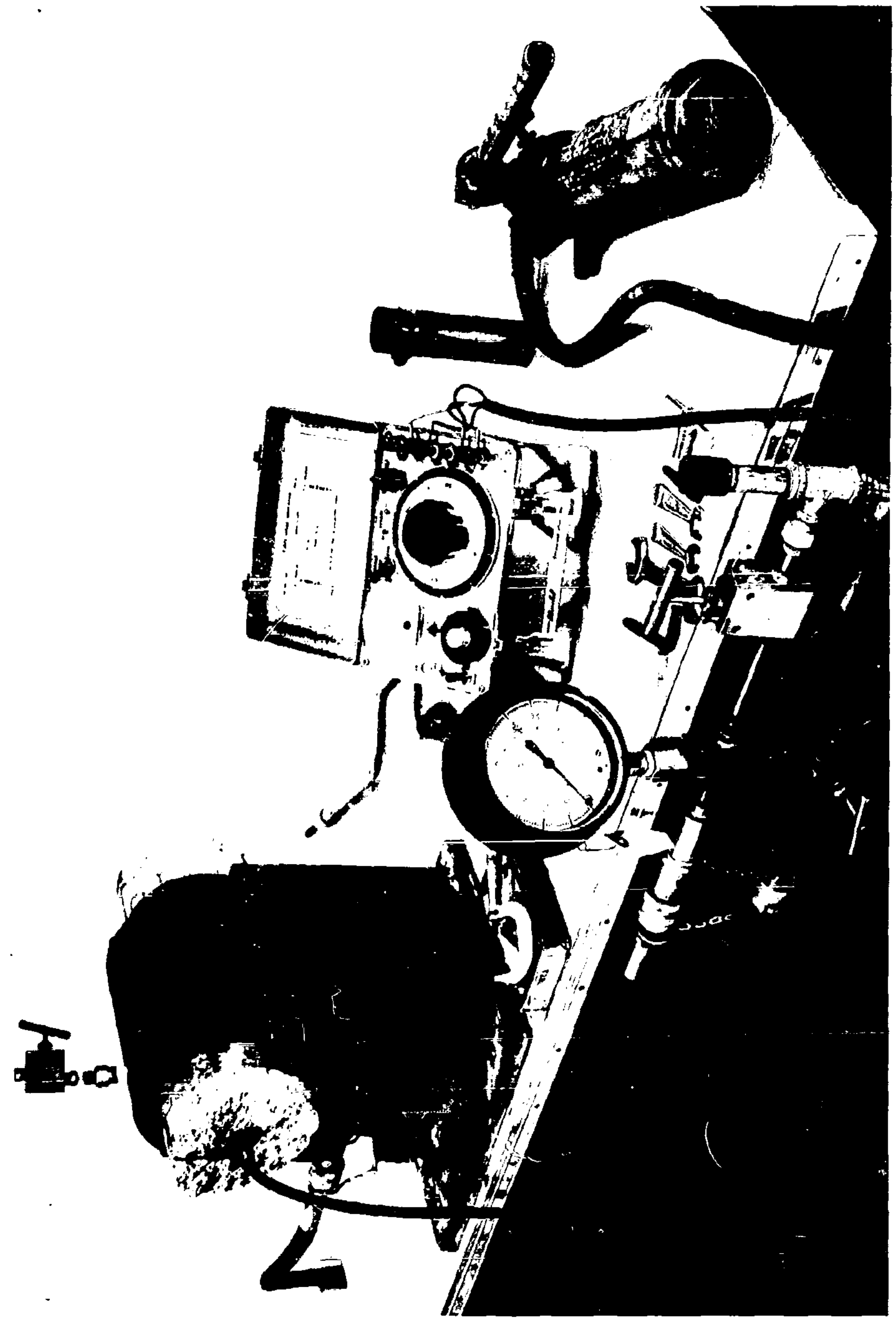

Photo. 9 Equipment for Modulus of Elasticity Testing

TECHNICAL REPORTS SCHEDULED FOR ISSUANCE BY AGENCIES PARTICIPATING IN PROJECT SHOAL

AEC REPORTS

Agency	Report No.	Project No.	Subject or Title
NBM	VUF-1001	33.2	Geological, Geophysical and Hydrological Investigations of the Sand Springs Range, Fairview Valley and Fourmile Flat, Churchill County, Nevada
SC	VUF-1002	40.5	Seismic Measurements at Sandia Stations
SC	VUF-1003	45.3	Hydrodynamic Yield Measurements
SC	VUF-1004	45.5	Device Support, Arming, Stemming and Yield Determination
SC	VUF-1005	45.6	Radiological Safety
EG&G	VUF-1006	60.4	Final Timing and Firing Report - Final Photo Report
USBM-PRC	*		Subsurface Fracturing From Shoal Nuclear Detonation
USWB	VUF-1008		Weather and Surface Radiation Prediction
USPHS	VUF-1009		Off-Site Surveillance
USBM	VUF-1010		Structural Survey of Private Mining Properties
USC&GS	VUF-1011		Seismic Safety Net
REECo	VUF-1012		On-Site Health and Safety Report

Agency	Report No.	Project No.	Subject or Title
RFB, Inc.	VUF-1013		Analysis of Shoal Data on Ground Motion and Containment
H-NSC	VUF-1014		Shoal Post-Shot Hydrologic Safety Report
H&N	VUF-1015		Pre-Shot and Post-Shot Structure Survey
H&N	VUF-1016		Test of Dribble-Type Structures
FAA	VUF-1017		Federal Aviation Agency Airspace Advisory
		DOD REPORTS	
SC	VUF-2001	1.1	Free Field Earth Motions and Spalling Measurements in Granite
SC	VUF-2002	1.2	Surface Motion Measurements Near Surface
** USC&GS	VUF-2300	1.4	Strong Motion Seismic Measurements
LPI	VUF-2600	1.6	In-Situ Stress in Granite
WES	VUF-2700	9.1	Grouting Support
** STL	VUF-2400	1.7	Shock Spectrum Measurements
SRI	VUF-3001	7.5	Investigation of Visual and Photographic On-Site Techniques
SRI	VUF-3002	7.6	Local Seismic Monitoring - Vela CLOUD GAP Program

TI	VUF-3003	7.8	Surface and Subsurface Radiation Studies
USGS	VUF-3004	7.9	Physical and Chemical Effects of the Shoal Event
ITEK	VUF-3005	7.10	Airborne Spectral Reconnaissance
BR Ltd.	VUF-3006	7.15	The Mercury Method of Identification and Location of Underground Nuclear Sites
NRDL	VUF-3007	7.16	Multi-Sensor Aerial Reconnaissance of an Underground Nuclear Detonation
GIMRADA	VUF-3008	7.17	Stereophotogrammetric Techniques for On-Site Inspection
ISOTOPES	VUF-3009	7.19	Detection in Surface Air of Gaseous Radionuclides from the Shoal Underground Detonation
*** USC&GS		8.1	Microearthquake Monitoring at the Shoal Site
**** GEO-TECH		8.4	Long-Range Seismic Measurements

* This is a Technical Report to be issued as PNE-3001 which will receive TID-4500 category UC-35 Distribution "Nuclear Explosions-Peaceful Applications"

** Project Shoal results are combined with other events, therefore, this report will not be printed or distributed by DTIE

*** Report dated March 1964 has been published and distributed by USC&GS

**** Report dated December 9, 1963, DATDC Report 92, has been published and distributed by UED

LIST OF ABBREVIATIONS FOR TECHNICAL AGENCIES

BR Ltd.	Barringer Research Limited Rexdale, Ontario, Canada
EG&G	Edgerton, Germeshausen & Grier, Inc. Boston, Massachusetts Las Vegas, Nevada Santa Barbara, California
FAA	Federal Aviation Agency Los Angeles, California
GEO-TECH	Geo Technical Corporation Garland, Texas
GIMRADA	U. S. Army Geodesy, Intelligence and Mapping Research and Development Agency Fort Belvoir, Virginia
H-NSC	Hazleton-Nuclear Science Corporation Palo Alto, California
H&N, Inc.	Holmes & Narver, Inc. Los Angeles, California Las Vegas, Nevada
ISOTOPES	Isotopes, Inc. Westwood, New Jersey
ITEK	ITEK Corporation Palo Alto, California
LPI	Lucius Pitkin, Inc. New York, New York
NBM	Nevada Bureau of Mines University of Nevada, Reno, Nevada
NRDL	U. S. Naval Radiological Defense Laboratory San Francisco, California
REECo	Reynolds Electrical & Engineering Co., Inc. Las Vegas, Nevada
SC	Sandia Corporation Albuquerque, New Mexico
SRI	Stanford Research Institute Menlo Park, California

RFB, Inc.	R. F. Beers, Inc. Alexandria, Va.
STL	Space Technology Laboratories, Inc. Redondo Beach Park, California
TI	Texas Instruments, Inc. Dallas, Texas
USBM	U. S. Bureau of Mines Washington, 25, D. C.
USBM-PRC	U. S. Bureau of Mines Bartlesville Petroleum Research Center Bartlesville, Oklahoma
USC&GS	U. S. Coast and Geodetic Survey Las Vegas, Nevada
USGS	U. S. Geologic Survey Denver, Colorado
USPHS	U. S. Public Health Service Las Vegas, Nevada
USWB	U. S. Weather Bureau Las Vegas, Nevada
WES	Waterways Experiment Station U. S. Army Engineers Vicksburg, Mississippi